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NATIONAL DAM SAFETY PROGRAM. VALLE LAKE DAM (MO 30438), MISSISS--ETC(U)
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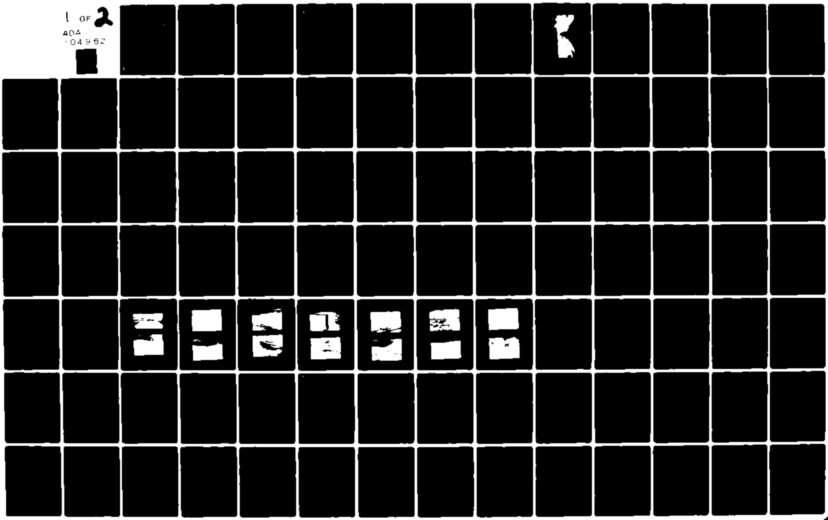

LEVEL II

MISSISSIPPI · KASKASKIA · ST. LOUIS BASIN

VALLE LAKE DAM
JEFFERSON COUNTY, MISSOURI
MO. 30438

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

United States Army
Corps of Engineers
...Serving the Army
...Serving the Nation

St. Louis District

PREPARED BY: U. S. ARMY ENGINEER DISTRICT, ST. LOUIS

FOR: STATE OF MISSOURI

JULY 1981

UNCLASSIFIED

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This report was prepared under the National Program of Inspection of Non-Federal Dams. This report assesses the general condition of the dam with respect to safety, based on available data and on visual inspection, to determine if the dam poses hazards to human life or property.

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DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
210 TUCKER BOULEVARD, NORTH
ST. LOUIS, MISSOURI 63101

REPLY TO
ATTENTION OF

SUBJECT: Valle Lake Dam (Mo. 30438) Phase I Inspection Report

This report presents the results of field inspection and evaluation of the Valle Lake Dam (Mo. 30438).

It was prepared under the National Program of Inspection of Non-Federal Dams.

This dam has been classified as unsafe, non-emergency by the St. Louis District as a result of the application of the following criteria:

a. The combined capacity of the spillways will not pass 50 percent of the Probable Maximum Flood without overtopping the dam.

b. Overtopping of the dam could result in failure of the dam.

c. Dam failure significantly increases the hazard to loss of life downstream.

SUBMITTED BY: SIGNED — 21 JUL 1981
Chief, Engineering Division — Date

APPROVED BY: SIGNED — 21 JUL 1981
Colonel, CE, Commanding — Date

A

VALLE LAKE DAM
JEFFERSON COUNTY, MISSOURI

MISSOURI INVENTORY NO. 30438

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED BY
PRC CONSOER TOWNSEND, INC.
ST. LOUIS, MISSOURI
AND
PRC ENGINEERING CONSULTANTS, INC.
ENGLEWOOD, COLORADO
A JOINT VENTURE

UNDER DIRECTION OF
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
FOR
GOVERNOR OF MISSOURI

JULY 1981

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam:	Valle Lake Dam, Missouri Inventory No. 30438
State Located:	Missouri
County Located:	Jefferson
Stream:	Unnamed tributary of Joachim Creek
Date of Inspection:	March 6, 1981

Assessment of General Condition

Valle Lake Dam was inspected by the engineering firms of PRC Consoer Townsend, Inc. of St. Louis, Missouri, and PRC Engineering Consultants, Inc. of Englewood, Colorado, (A Joint Venture) in accordance with the U.S. Army Corps of Engineers "Recommended Guidelines for Safety Inspection of Dams" and additional guidelines furnished by the St. Louis District of the Corps of Engineers. Based upon the criteria in the guidelines, the dam is in the high hazard potential classification, which means that loss of life and appreciable property loss could occur in the event of failure of the dam. Located within the estimated damage zone of five miles downstream of the dam are 16 dwellings, several buildings, and a county highway (Highway V), which parallels Joachim Creek, all of which may be subjected to flooding, with possible damage and/or destruction, and possible loss of life. Valle Lake Dam is in the small size classification since it is 38.8 feet high and has a maximum reservoir impoundment of 681 acre-feet.

The inspection and evaluation indicate that the spillway system of Valle Lake Dam does not meet the criteria set forth in the

guidelines for a dam having the above size and hazard potential. Valle Lake Dam being a small size dam with a high hazard potential is required by the guidelines to pass from one-half of the Probable Maximum Flood to the Probable Maximum Flood without overtopping the dam. Considering the possibility of loss of life and the destruction of property downstream of the dam, the PMF is considered the appropriate spillway design flood for Valle Lake Dam. The Probable Maximum Flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the region. It was determined that the reservoir/spillway system can accommodate approximately 30 percent of the Probable Maximum Flood without overtopping the dam. The evaluation also indicates that the reservoir/spillway system will accommodate the one-percent chance flood (100-year flood) without overtopping the dam.

The overall condition of the dam and the spillways appears to be fair; however, the one area of seepage and the two areas of possible seepage observed along the downstream slope jeopardize the safety of the dam and will require prompt attention. Other deficiencies noted by the inspection team, which will require remedial measures, included: the deterioration of the concrete of the principal spillway drop inlet; a few minor problems observed in the principal and emergency spillways consisting of debris, joint displacements, undermining, and voids in the concrete lining; the minor erosion of the upstream slope due to wave action; a lack of a maintenance schedule; and there also exists a need for periodic inspection by a qualified engineer. The lack of seepage and stability analyses on record is also a deficiency that should be corrected.

It is recommended that the owner take action to correct or control the deficiencies described above.

Walter G. Shifrin, P.E.

STATE OF MISSOURI
WALTER
G SHIFRIN
NUMBER
E-8834
REGISTERED PROFESSIONAL ENGINEER

Overview of Valle Lake Dam

NATIONAL DAM SAFETY PROGRAM

VALLE LAKE DAM, I.D. No. 30438

TABLE OF CONTENTS

TABLE OF CONTENTS

(Continued)

TABLE OF CONTENTS

(Continued)

LIST OF PLATES

APPENDICES

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

VALLE LAKE DAM, Missouri Inv. No. 30438

SECTION 1: PROJECT INFORMATION

1.1 General

a. Authority

The Dam Inspection Act, Public Law 92-367 of August, 1972, authorizes the Secretary of the Army, through the Corps of Engineers, to initiate a national program of dam inspections. Inspection for Valle Lake Dam was carried out under Contract DACW 43-81-C-0063 between the Department of the Army, St. Louis District, Corps of Engineers, and the engineering firms of PRC Consoer Townsend, Inc. of St. Louis, Missouri, and PRC Engineering Consultants, Inc. of Englewood, Colorado, (A Joint Venture).

b. Purpose of Inspection

The visual inspection of Valle Lake Dam was made on March 6, 1981. The purpose of the inspection was to make a general assessment as to the structural integrity and operational adequacy of the dam embankment and its appurtenant structures.

c. Scope of Report

This report summarizes available pertinent data relating to the project, presents a summary of visual observations made during the field inspection, presents an assessment of hydrologic and hydraulic conditions at the site and of the structural adequacy

of the various project features, and assesses the general condition of the dam with respect to safety.

Subsurface investigations, laboratory testing and detailed analyses were not within the scope of this study. No warranty as to the absolute safety of the project features is implied by the conclusions presented in this report.

It should be noted that in this report reference to left or right abutments is viewed as looking downstream. Where left abutment or left side of the dam is used in this report, this also refers to the west abutment or side, and right to the east abutment or side.

d. Evaluation Criteria

The inspection and evaluation of the dam is performed in accordance with the U.S. Army Corps of Engineers "Recommended Guidelines for Safety Inspection of Dams" and additional guidelines furnished by the St. Louis District office of the Corps of Engineers for Phase I Dam Inspection.

1.2 Description of the Project

a. Description of Dam and Appurtenances

The following description is based upon observations and measurements made during the visual inspection and conversations with Mr. Edwin Lucas, the previous owner of the dam. No design or "as-built" drawings for the dam or appurtenant structures were available.

The dam is a rolled, earthfill structure with a core trench excavated to solid bedrock, according to Mr. Lucas. Mr. Lucas also stated that the embankment was constructed of mostly a clay material with any gravel material encountered during the

construction of the dam being placed in the downstream shell of the dam. The alignment of the dam is straight between earth abutments. A plan and elevation of the dam are shown on Plate 4 and Photos 1 through 3 show views of the dam. The top of dam has a length of 820 feet between the right abutment and the emergency spillway. The minimum elevation of the top of dam was found to be 689.0 feet above mean sea level (M.S.L.) at two points on the dam located at 300 and 500 feet from the right abutment. Between the two points, the top of dam was surveyed to be level. The right and left ends of the dam were determined to be 1.1 feet and 0.5 feet higher in elevation, respectively, than the minimum top of dam elevation. The embankment has a top width of 28 feet and a maximum structural height of 38.8 feet. The downstream slope was measured to be 1 vertical to 3 horizontal (1V to 3H). Riprap was placed on the upstream slope near the normal pool elevation and extended to well below the water surface. The placement of the riprap created a small bench on the upstream slope. A sand and gravel berm was also placed on the upstream slope on top of the riprap (see Photo 1). The berm extended from the left side of the dam to approximately the center of the dam. The upstream slope, the sand and gravel berm, and the riprap wave protection were all measured to be 1V to 2.5H above the water surface on the day of the inspection.

There are two spillways at this damsite that are referred to in this report as the principal and the emergency spillway. The principal spillway is a concrete drop inlet structure with a 21-inch diameter CMP outlet located at the left abutment (see Photo 7). The inside dimensions of the inlet as measured in the field are a width of 4.3 feet, a length of 4.5 feet and a height of 7.7 feet. The front- and side-walls are 12 inches thick and the backwall is integral with a concrete retaining wall which forms part of the emergency spillway. The front of the structure for the top 3.5 feet is open from sidewall to sidewall. The opening is 4.3 feet wide and has a minimum crest elevation of 680.9 feet above M.S.L. The sidewalls adjacent to the front opening are fitted with stoplog slots. Stoplogs can be inserted raising the crest of the inlet to

an assumed elevation of 684.0 feet above M.S.L. The top of the inlet is fitted with a metal grate with a crest elevation of 684.4 feet above M.S.L. The inlet of the CMP is flush with the inside face of the drop structure backwall. The invert of the pipe is at the floor of the drop structure, 4.2 feet below the minimum crest elevation of the drop structure. The CMP leads under the emergency spillway and outlets onto a concrete pad. The discharge channel is lined with bedrock (see Photo 10). The alignment of the channel begins to make a 90 degree bend just downstream of the outlet of the principal spillway (see Photo 9) and then parallels the axis of the dam until the channel intersects the downstream channel. A training berm runs along the right side of the channel, protecting the dam (see Photo 3). Discharge through the spillway joins the downstream channel about 150 feet downstream of the toe of the dam.

The emergency spillway is a trapezoidal shaped broad-crested weir located on the left abutment (see Photo 6). The leading edge of the weir is a vertical retaining wall which also provides the backwall of the drop inlet principal spillway. The crest of the weir is paved with an asphalt access road that extends across the entire length of the dam. The crest length of the spillway is 135 feet with an invert elevation of 684.5 feet above M.S.L. The spillway side slopes are 1V to 16H and 1V to 9H on the left and right side, respectively. The back slope of the spillway proceeds over the downstream side of the weir and is surfaced with a mixture of riprap and concrete (see Photo 8). Discharge through the emergency spillway enters the discharge channel of the principal spillway, which is described above. An upstream training berm was placed perpendicular to the axis of the dam on the right side of the approach channel to both spillways to protect the dam (see Photo 2).

No low-level outlet or outlet works were provided for this dam.

b. Location

Valle Lake Dam is located in Jefferson County in the State of Missouri on an unnamed tributary of Joachim Creek. The location of the dam on the 7.5 minute series of the U.S. Geological Survey maps is found in the northeast quadrant of Section 31 of Township 39 North, Range 5 East, of the Vineland, Missouri Quadrangle Sheet (Advance Print, see Plate 2). The dam is located approximately six miles southeast of De Soto (see Plate 1).

c. Size Classification

The reservoir impoundment of Valle Lake Dam is less than 1,000 acre-feet but more than 50 acre-feet, which would classify it as a "small" size dam. The maximum height of the dam is less than 40 feet and greater than 25 feet, which also classifies it as a "small" size dam. The size classification is determined by either the storage or height, whichever gives the larger size category. Therefore, the size classification is determined to fall within the "small" category, according to the "Recommended Guidelines for Safety Inspection of Dams" by the U.S. Department of the Army, Office of the Chief Engineer.

d. Hazard Classification

The dam has been classified as having a "high" hazard potential in the National Inventory of Dams, on the basis that in the event of failure of the dam or its appurtenances, excessive damage could occur to downstream property, together with the possibility of the loss of life. From a visual inspection of the downstream area, our findings concur with this classification. Located within the estimated damage zone, which extends approximately five miles downstream of the dam, are at least 16 dwellings, several buildings, and a county highway (Highway V), which parallels Joachim Creek (see Photos 13 and 14).

e. Ownership

Valle Lake Dam is privately owned by the Valle Lake Property Owners Association. The Valle Lake Property Owners Association is governed by a board of three trustees. The mailing address is as follows: Valle Lake Property Owners Association, c/o Mr. Raymond Sherer, Trustee, Valle Lake, Route 3, De Soto, Missouri, 63020.

f. Purpose of Dam

The purpose of the dam is to impound water for recreational use as a private lake.

g. Design and Construction History

According to Mr. Raymond Sherer, the dam was built by the Lucas Construction Company in 1955. Mr. and Mrs. Edwin Lucas were the owners of the construction company and the original owners of Valle Lake Dam.

Mr. Edwin Lucas indicated in a telephone conversation that there were no drawings or specifications for the dam. According to Mr. Lucas, gravel encountered during the construction was placed in the downstream shell of the dam. Sound bedrock was encountered about five feet below the natural ground at the damsite. A core trench was exacaveted along the axis of the dam to the solid bedrock. The dam was constructed using bulldozers and scrapers.

h. Normal Operational Procedures

The water surface level of the reservoir can be controlled by the principal spillway drop inlet, which can lower the reservoir 3.1 feet below the normal water surface elevation. Stoplogs are inserted or removed from the stoplog slots of the drop inlet structure to raise or lower the reservoir water surface. The reservoir water surface is normally lowered between November and March in order to protect boat docks from ice damage and to release excess runoff from spring storms. The reservoir surface is then raised for the remainder of the year and allowed to remain as high as possible.

1.3 Pertinent Data

a. Drainage Area (square miles): 2.98

b. Discharge at Damsite

Estimated experienced maximum flood (cfs): 400

Estimated ungated spillway capacity with reservoir at top of dam elevation (cfs): 5,030

c. Elevation (Feet above M.S.L.)

Top of dam (minimum): 689.0

Principal Spillway crest (without stoplogs): 680.9

Principal Spillway crest (with stoplogs): . . . . 684.0 (assumed)*

Emergency Spillway crest: 684.5

Normal Pool: 684.0

Maximum Experienced Pool: 685.5

Observed Pool: 681.3

d. Reservoir

Length of pool with water surface at top of dam elevation (feet): 3,400

e. Storage (Acre-Feet)

Top of dam (minimum): 681

Principal Spillway crest (without stoplogs): 313

Principal Spillway crest (with stoplogs): 431

Emergency Spillway crest: 453

Normal Pool: 431

Maximum Experienced Pool: 504

Observed Pool: 328

f. Reservoir Surfaces (Acres)

Top of dam (minimum): 57.5

Principal Spillway crest (without stoplogs): 34.0

Principal Spillway crest (with stoplogs): 42.5

Emergency Spillway crest:	44.0
Normal Pool:	42.5
Maximum Experienced Pool:	47.0
Observed Pool:	35.0

g. Dam

Type:	Rolled, Earthfill
Length:	820 feet
Structural Height:	38.8 feet
Hydraulic Height**:	38.8 feet
Top width:	28 feet
Side slopes:	
Downstream	1V to 3H (measured)
Upstream	1V to 2.5H (from the top of dam to the elevation of the water surface on the day of the inspection; also applies to the sand and gravel berm and the riprap wave protection).
Zoning:	Clay embankment with gravel placed in the downstream shell.
Impervious core:	N.A.
Cutoff:	A core trench excavated to bedrock, according to Mr. Lucas
Grout curtain:	None
Volume:	99,100 cu.yds. (estimated)

h. Diversion and Regulating Tunnel. . . . .None

i. Spillway

Type:	
Principal Spillway	Concrete drop inlet, controlled with stoplogs
Emergency Spillway	Broad-crested weir paved with asphalt, uncontrolled

Location:

Principal Spillway . . . . Left abutment

Emergency Spillway . . . . Left abutment

Length of crest:

Principal Spillway . . . . 4.3 feet (front inlet)
13.3 feet (top inlet)

Emergency Spillway . . . . 135 feet

Crest Elevation (feet above M.S.L.):

Principal Spillway (front inlet) . . 680.9 (without stoplogs)
(front inlet) . . 684.0 (with stoplogs) (assumed)*
(top inlet) . . . 684.4

Emergency Spillway 684.5

j. Regulating Outlets . . None

* The elevation of the crest of the principal spillway with stoplogs in place is assumed to be the elevation of the reservoir as shown on the U.S.G.S. Vineland, Missouri Quadrangle topographic map. The elevations of other features of the dam are obtained by using this elevation and field measurements.

** The hydraulic height of the dam is the vertical distance from the lowest point on the downstream toe to the top of dam or the maximum water surface, if below the top of dam.

SECTION 2: ENGINEERING DATA

2.1 Design

No design drawings or data are available for Valle Lake Dam.

2.2 Construction

The dam was built by the Lucas Construction Company of De Soto, Missouri in 1955. No construction records or data are available relative to the construction of the dam, other than the construction history given in Section 1.2g.

2.3 Operation

No documented operational records or data are available for the dam.

2.4 Evaluation

a. Availability

The availability of engineering data is poor and consists only of the State Geological Maps, a general soil map of the State of Missouri published by the Soil Conservation Service, and U.S.G.S. Quadrangle Sheets.

b. Adequacy

The lack of engineering data did not allow for a definitive review and evaluation. The conclusions presented in this report are based on field measurements, past performance and present condition of the dam. The available data including the field

measurements taken by the field inspection team are considered adequate to evaluate the hydraulic and hydrologic capabilities of the dam. Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency. These seepage and stability analyses should be performed for appropriate loading conditions (including earthquake loads) and made a matter of record.

c. Validity

No valid engineering data pertaining to the design or construction of the dam were available.

SECTION 3: VISUAL INSPECTION

3.1 Findings

a. General

A visual inspection of the Valle Lake Dam was made on March 6, 1981. The following persons were present during the inspection:

Name	Affiliation	Disciplines
Mark Haynes, P.E.	PRC Engineering Consultants, Inc.	Soils
Jerry Kenny	PRC Engineering Consultants, Inc.	Hydraulics and Hydrology
James Nettum, P.E.	PRC Engineering Consultants, Inc.	Civil-Structural and Mechanical
Razi Quraishi, R.P.G.	PRC Engineering Consultants, Inc.	Geology
John Lauth, P.E.	PRC Consoer Townsend, Inc.	Civil-Structural

Specific observations are discussed below.

b. Dam

The overall condition of the dam appears to be fair; however, a few items of concern were observed and are described below.

The top of dam supports an asphalt access road used by the local residents to gain access to their homes. The road provides excellent protection for the top of dam. No depressions or cracks indicating a settlement of the embankment were apparent. The variation in elevation across the top of dam did not appear to be due to an instability of the embankment. No significant deviation in the horizontal alignment was apparent. According to Mr. Sherer, the dam has never been overtopped and no evidence indicating the contrary was observed.

The upstream slope was protected from erosion due to wave action by riprap consisting of dolomite stones ranging in size from three inches to three feet in diameter. The riprap appeared to be providing adequate protection; however, some very minor wave erosion was observed above the riprap. No deterioration of the riprap was observed. The sand and gravel berm on the slope appears to have been added after the construction of the dam and has little or no effect on the stability or safety of the dam. The slope above the riprap supported a sparse growth of grass. The grass cover, however, did appear to provide adequate protection against erosion due to surface runoff for the slope for no erosion was observed. No bulges, depressions or cracks indicating an instability of the embankment or foundation were observed on the slope. The effect of drawing down the reservoir has had no apparent effect on the stability of the embankment.

The downstream slope is adequately protected against surface runoff by a well maintained grass cover (see Photo 3). No trees were growing on the slopes. No bulges, depressions or cracks indicative of a major slope movement were apparent. However, near

the top of the dam, the slope steepens to approximately 1V to 1.5H, which would indicate that the slope could have failed in this area in the past. Nevertheless, it is felt that the steepness of the slope is due to the placement of fill material on top of the dam to facilitate the access road. The telephone poles located on the slope near the top of dam show no displacement in their vertical alignment, which would also indicate that no slope movement has taken place.

An area of seepage and two areas of possible seepage were observed along the downstream slope of the dam. The area of seepage was observed along the right abutment/embankment contact. Flowing water was observed at about midway up the abutment contact. The flow rate was less than 0.5 gallons per minute and the discharge was clear. It was undetermined whether the seepage was through the embankment or cracks in the foundation bedrock. Bedrock was exposed in this area.

One of the areas of possible seepage was observed just downstream of the embankment along the left abutment/embankment contact (see Photo 5). No measurable flow of water was observed in the area; however, the ground was boggy and standing water was observed in the area. It was unknown whether the source of the moisture in this area is due to seepage through the embankment or the foundation bedrock or due to recent rainstorms in the area. Bedrock was also exposed in this area.

The second area of possible seepage was observed near the center of the dam. The area was approximately 150 feet wide and extended from the toe of the dam up to about mid-height of the dam. The area was characterized by moist, boggy ground, standing water and several scarps (see Photo 4). No measurable flow of water was observed in the area. The scarps were measured to be up to six-feet wide and one-foot deep. The source of the moisture in this area is unknown; however, due to the location of the area, it is most likely due to seepage through the embankment. This would cause the

embankment material to be weakened, which could have caused the scarps in the area. Cattails were also observed which would indicate that moisture is generally present in the area.

Both abutments slope gently upward from the dam. No instabilities were observed on either abutment. One erosional gully was observed along the downstream, right abutment/embankment contact; however, the erosion is felt to have no bearing on safety of the dam for the erosion gully has been eroded to bedrock and is in no danger of eroding any further. No erosion felt to be detrimental to the safety of the dam or abutment was apparent on the left abutment.

No evidence of burrowing animals was apparent on either the embankment or the abutments.

c. Project Geology and Soils

(1) Project Geology

The damsite is located on an unnamed tributary of Joachim Creek in the Salem Plateau section of the Ozark Plateaus Physiographic Province. Deep dissection of topography by major streams is one of the important characteristics of the Salem Plateau section. There is a wide distribution of dolomites and limestones in the Salem Plateau. Cuestaform topography is exhibited in this plateau section consisting of two major escarpments, namely the Crystal Escarpment and Burlington Escarpment. Deep dissection in dolomites and limestones is a major factor in the development of many springs in this area. A major component of surface discharge of water to the regional drainage is contributed by these springs.

The topography in the vicinity of the damsite is rolling to hilly with U- to V-shaped valleys. Elevations of the ground surface range from 1034 feet above M.S.L. nearly 2.4 miles southwest of the damsite to 684 feet above M.S.L. at the damsite. The

reservoir slopes are generally from six- to eight-degrees from horizontal. The reservoir slopes are stable. The area near the damsite is covered with residual soil deposits consisting of a reddish-brown and orangey-brown mottled, moderately plastic, silty clay with occasional rock fragments less than 1/4 inch in size.

The regional bedrock geology beneath the residual soil deposits in the damsite area as shown on the Geologic Map of Missouri (1979) (see Plate 6) are of the Ordovician age rocks consisting of Decorah Formation, St. Peter Sandstone, Powell Dolomite, Cotter Dolomite, Roubidoux Formation, and Gasconade Dolomite; and the Cambrian age rocks consisting of Eminence Dolomite, Potosi Dolomite, Lamotte Sandstone, and Franconia and Bonneterre Formations. The predominent bedrock underlying the residual soil deposits in the vicinity of the damsite are the Ordovician age rocks consisting of Powell Dolomite and Roubidoux Formation.

Outcroppings of Ordovician Powell Dolomite (light brown, fine grained, very hard, thinly bedded, slightly weathered dolomite) interbedded with shale are exposed in the downstream areas of both abutments and in the discharge channel of the spillways (see Photos 10 and 11).

No faults have been identified at the damsite. The closest trace of a fault to the damsite is the Ste. Genevieve fault system nearly two miles northeast of the damsite. The Ste. Genevieve fault had its last movement in the post-Pennsylvanian time. Thus, the fault system has no effect on the damsite.

No boring logs or construction reports are available that would indicate foundation conditions encountered during construction. Based on the visual inspection and conversations with Mr. Lucas, the embankment probably rests on the bedrock of the Ordovician Powell Dolomite with the core trench excavated to the bedrock. The dual spillway system rests on the residual soils of the left abutment.

(2) Project Soils

According to the "Missouri General Soil Map and Soil Association Description" published by the Soil Conservation Service, the materials in the general area of the dam belong to the soil series of Union-Goss-Gasconade-Peridge in the Ozark Border Association. The soils are basically formed from loess deposits and weathered bedrock. These soils vary from a slowly permeable silty clay to moderately permeable silt loam.

Material removed from the embankment slopes was a reddish-brown, moderately plastic, silty clay with traces of fine to medium sand and some 1/4 inch rock fragments. Based upon the Unified Soil Classification System, the soil would probably be classified as a CL. This is an impervious soil type, which generally has the following characteristics: a coefficient of permeability less than one foot per year, medium shear strength, and a high resistance to piping.

Material removed from the berm on the upstream slope was a poorly graded, silty, sand and gravel, which would probably be classified as a GM. The berm appeared to have been placed on the upstream slope after the construction of the dam. Therefore, the material used for the berm should not be considered to be representative of the material used in the embankment.

d. Appurtenant Structures

(1) Principal Spillway

The front edge of the right sidewall has partially disintegrated into the stoplog slot (see Photo 7). No reinforcement was evident. The remaining concrete surfaces appeared stable with minor weathering observed. A small amount of debris was seen in the bottom of the drop inlet; however, the spillway was operating on the day of the inspection. It appears that some repair work was done

around the inlet of the outlet pipe. The concrete surface was rough and the edge of the pipe was bent and rusting in places. There were several minor deformation of the pipe observed along the length of the conduit, which were probably caused by vehicular traffic during construction before sufficient backfill was placed over the pipe. There was some undermining around the edges of the outlet pad and a slight amount of differential settlement has taken place at the transverse joint in the middle of the pad. The discharge channel appeared stable resembling a natural stream with a series of pools, riffles and falls over the in situ bedrock.

(2) Emergency Spillway

The retaining wall at the leading edge of the spillway weir appeared sound, with no misalignment or damage to the concrete. The asphalt surface of the weir was smooth and stable. The back slope of the weir was irregular but appeared stable (see Photo 8). There were some voids in the concrete/riprap covering probably caused by flow down the slope. The variation in the color of the concrete indicates that the surfacing was done piecemeal.

(3) Outlet Works

No low-level outlet or outlet works were provided for this dam.

e. Reservoir Area

The reservoir water surface elevation at the time of the inspection was 681.3 feet above M.S.L. The reservoir has a normal water surface elevation of 684.0 feet above M.S.L. and a surface area of 42.5 acres at the normal water surface level.

The rim appeared to be stable with no erosional or stability problems observed (see Photo 12). The land around the reservoir slopes gently upward from the reservoir rim and is mostly wooded with grass slopes. Several houses are built around the reservoir rim. No evidence of excessive siltation was observed in the reservoir on the day of the inspection.

Four dams and reservoirs are located upstream of Valle Lake and were considered to be large enough to have an effect on the flood routing evaluation for Valle Lake Dam, as further discussed in Section 5 (see Plate 2). The four dams are named as follows: Lucas Dam (Mo. 30388); Lucas Lake Dam (Mo. 30454); Lower Valle Mines Dam (Mo. 30439); and Upper Valle Mines Dam (Mo. 30370).

f. Downstream Channel

The downstream channel near the dam is the natural streambed with approximate dimensions of four to five feet deep and 30 feet wide. Outside of the streambed, the downstream channel widens into a fairly wide flood plain (see Photo 13). The channel near the damsite was unobstructed.

3.2 Evaluation

The visual inspection did not reveal any conditions which were felt to constitute an unsafe condition at this time; however, the following condition does exist which warrants prompt attention.

The one area of seepage and the two areas of possible seepage observed along the downstream slope could have a detrimental effect on the structural stability of the dam. In the one area of measurable seepage, the flow rate of the seepage could possibly increase. This could cause piping of embankment material which could lead to the eventual failure of the embankment. In the two areas of possible seepage, if indeed the condition is due to seepage, it is possible the same condition as described above could occur with time.

The following conditions were observed which could adversely affect the dam in the future and will require maintenance within a reasonable period of time.

1. The disintegration of the principal spillway right sidewall does not presently affect the safety of the dam. Further deterioration will preclude the use of the stoplog slot and the reservoir regulation that the drop inlet provides will be lost. The complete failure of the wall could impair the functioning of the spillway and thus imperil the safety of the dam.

2. The amount of debris in the drop inlet of the principal spillway was not sufficient to obstruct the spillway. But, further accumulations of debris could block the spillway outlet and jeopardize the safety of the dam.

3. The undermining and joint displacement of the concrete outlet pad of the principal spillway do not appear to affect the structural integrity of the spillway in the present condition. Nonetheless, with time these conditions could worsen to the point where the safety of the spillway could be jeopardized.

4. The voids in the surfacing of the back slope of the emergency spillway did not appear to affect the spillway stability. But, with future flows through the emergency spillway, this condition could worsen and become a hazard to the stability of the spillway and access road.

5. The very minor wave erosion on the upstream slope does not appear to affect the stability of the dam in its present condition. However, continual erosion of the slope can only be detrimental to the structural integrity of the dam.

SECTION 4: OPERATIONAL PROCEDURES

4.1 Procedures

Valle Lake Dam is used to impound water for recreational use as a private lake. Normal operation procedures for the dam and reservoir are outlined in Section 1.2h. The water level is kept as high as possible. The water surface elevation is controlled by rainfall, runoff, evaporation and by the elevation of the principal spillway crest.

4.2 Maintenance of Dam

Each property owner in the Valle Lake Property Owners Association is assessed a fee for maintenance of the lake and dam property. The grass is periodically mowed from the downstream slope and the upper part of the upstream slope. The access road across the top of dam is also periodically resurfaced.

There have not been any major repairs made to the dam since its original construction.

4.3 Maintenance of Operating Facilities

The principal spillway is the only operating facility at the damsite, which requires maintenance. The maintenance of the spillway appears to be somewhat lacking due to the observed damage to the drop inlet structure. Nevertheless, the spillway is operable and is capable of functioning as original intended.

4.4 Description of Any Warning System in Effect

The inspection team is not aware of any warning system in use at the damsite, such as an electrical warning system or a manual notification plan.

4.5 Evaluation

The maintenance at Valle Lake Dam appears to be adequate; however, the remedial measures described in Section 7 should be undertaken to improve the condition of the dam.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 Evaluation of Features

a. Design

No hydrologic and hydraulic design data are available for Valle Lake Dam. The sizes of physical features utilized to develop the stage-outflow relation for the spillway and overtopping of the dam were prepared from field notes and sketches prepared during the field inspection. The reservoir elevation-area data were based on the U.S.G.S. Vineland, Missouri Quadrangle topographic map (Advance Print, 7.5 minute series). The spillway and overtop release rates and the reservoir elevation-area data are presented in Appendix B.

The hydrologic soil group of the watershed was determined from information available in the U.S.D.A. Soil Conservation Service publication "Missouri General Soil Map and Soil Association Descriptions", 1979. The Probable Maximum Precipitation (PMP) used to determine the Probable Maximum Flood (PMF) was determined by using the U.S. Weather Bureau publication "Hydrometeorological Report No. 33" (April 1956). The 100-year and the 10-year floods were derived from the 100-year and the 10-year rainfall, respectively, of Ste. Genevieve, Missouri.

b. Experience Data

Records of reservoir stage or spillway discharge are not maintained for this site. However, according to Mr. Sherer, the maximum reservoir level was approximately 12 inches above the crest of the spillway.

c. Visual Observations

Observations made of the spillways during the visual inspection are discussed in Section 3.1d and evaluated in Section 3.2.

d. Overtopping Potential

Both the Probable Maximum Flood and one-half of the Probable Maximum Flood, when routed through the reservoir, resulted in overtopping of the dam. The peak inflows of the PMF and one-half of the PMF are 21,989 cfs and 11,025 cfs, respectively. The peak outflow discharges for the PMF and one-half of the PMF are 21,173 cfs and 9,798 cfs, respectively. The maximum capacity of the spillway just before overtopping the dam is 5,030 cfs. The PMF overtopped the dam by 2.92 feet and one-half of the PMF overtopped the dam by 1.37 feet. The total duration of flow over the dam is 4.67 hours during the occurrence of the PMF and 1.25 hours during one-half of the PMF. The spillway/reservoir system of Valle Lake Dam is capable of accommodating a flood equal to approximately 30 percent of the PMF just before overtopping the dam and will also accommodate the one-percent chance flood (100-year flood) without overtopping the dam. The analysis of Valle Lake Dam included the hypothetical breach of the four upstream dams (Lucas Dam (Mo. 30388), Lucas Lake Dam (Mo. 30454), Lower Valle Mines Dam (Mo. 30439), and Upper Valle Mines Dam (Mo. 30370)) for those floods during which the dams were overtopped.

The surface soils on the embankment consists of a silty clay. The emergency spillway and the top of dam are covered by an asphalt road and the downstream slope has a good cover of grass. Nevertheless, the dam will be overtopped by approximately three feet during the occurrence of the PMF, which can cause severe erosion to the embankment due to the high velocity of flow on its downstream slope and could lead to the eventual failure of the dam. The relatively wide top width of 28 feet and the moderate downstream

slope of 1V to 3H in combination with the grass cover on the downstream slope and the asphalt pavement on top of the dam will have, however, some effect on reducing the possibility of a dam failure due to overtopping of the dam. The maximum velocity of flow in the emergency spillway and the discharge channel during the PMF will be about ten ft/sec, which could also cause some erosion in the spillway channel due to the high velocity of flow. Nevertheless, any erosion in the emergency spillway channel and the discharge channel will have little effect on the safety of the dam, if any at all.

The failure of the dam could cause extensive damage to the property downstream of the dam and possible loss of life. The estimated damage zone extends approximately five miles downstream of the dam. Located within the damage zone are at least 16 dwellings, several buildings, and a county highway (Highway V), which parallels Joachim Creek.

SECTION 6: STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observations

There were no major signs of settlement or distress observed on the embankment or foundation during the visual inspection. The stability of the dam does not appear to be in jeopardy at this time; however, the one area of seepage and the two areas of possible seepage observed along the downstream slope could be detrimental to the stability of the embankment, but they do not appear to constitute an unsafe condition at this time. The minor wave erosion on the upstream slope above the riprap protection does not appear to endanger the structural stability of the embankment in its present condition; however, continual erosion of the slope could be detrimental to the embankment. In the absence of seepage and stability analyses, no quantitative evaluation of the structural stability can be made.

The structural stability of the principal spillway appears to be questionable due to the disintegration of the drop inlet wall. However, this damage does not constitute an unsafe condition at this time. The emergency spillway appeared to be structurally stable with the exception of the voids on the back slope caused by erosion. The principal spillway was partially obstructed by debris on the day of the inspection, however, it was operating. The emergency spillway was unobstructed and appeared to be able to function properly.

b. Design and Construction Data

No design computations pertaining to the embankment were uncovered during the report preparation phase. Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available. No embankment or foundation soil parameters were available for carrying out a conventional stability analysis on the embankment. No construction data or specifications relating to the degree of embankment compaction were available for use in a stability analysis.

c. Operating Records

No documented operating records are available relating to the stability of the dam or appurtenant structures. However, the reservoir is lowered in the winter months and raised in the spring months by removing or replacing the stoplogs in the principal spillway. The lowering of the reservoir by these means should not have any effect on the stability of the dam due to the small difference between the normal pool elevation and the principal spillway crest elevation with the stoplogs removed. The water level on the day of inspection was 2.7 feet below the normal pool elevation.

d. Post Construction Changes

No post construction changes to the embankment are known to exist that will affect the structural stability of the dam.

e. Seismic Stability

The dam is located in Seismic Zone 2, as defined in the "Recommended Guidelines for Safety Inspection of Dams" as prepared by the Corps of Engineers (see Plate 9). Seismic Zone 2 is characterized by a moderate earthquake hazard. An earthquake of the magnitude that would be expected in Seismic Zone 2 should not cause significant distress to a well designed and constructed earth dam.

Available literature indicates that no active faults exist near the vicinity of the damsite. The maximum recorded historic magnitude earthquake in the immediate vicinity of the damsite was the July 21, 1967 event of magnitude 4.4 located at a distance of 36 miles south of the damsite. This event cannot be correlated with known tectonic structure and is considered to probably be related to the release of accumulated residual strain along a buried pre-Quaternary fault. The attenuation of this event to the damsite would produce a peak ground acceleration of less than 0.05g which would not produce a significant seismic impact on the dam.

SECTION 7: ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment

The assessment of the general condition of the dam is based upon available data and the visual inspection. Detailed investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

It should be realized that the reported condition of the dam is based upon observations of field conditions at the time of the inspection along with data available to the inspection team.

It is also important to realize that the condition of a dam depends upon numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be assurance that an unsafe condition could be detected.

a. Safety

The spillway capacity of Valle Lake Dam is found to be "Seriously Inadequate". The spillway/reservoir system will accommodate about 30 percent of the PMF without overtopping the dam. If the dam is overtopped, the safety of the embankment would be in jeopardy due to the susceptibility of the embankment materials to erosion. High velocity of flow on the downstream slope of the dam could cause excessive erosion and eventually lead to a failure of the dam. The spillway system could also receive some damage during the occurrence of a PMF.

The overall condition of the dam and apurtenant structures appears to be fair; however, the one area of seepage and the two areas of possible seepage jeopardize the safety of the dam and do warrant prompt attention. A quantitative evaluation of the safety of the embankment could not be made in view of the absence of seepage and stability analyses. The present embankment and appurtenant structures, however, appear to have performed satisfactorily since their construction. The dam has never been overtopped, according to Mr. Sherer, and no evidence indicating the contrary was observed. The safety of the dam can only be improved if the deficiencies described in Section 3.2 are properly corrected as described in Section 7.2b.

b. Adequacy ot Information

The conclusions presented in this report are based upon field measurements, past performance and the present condition of the dam. Documented information on the design hydrology, hydraulic design, operation, and maintenance of the dam was not available. Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency.

c. Urgency

The items recommended in paragraph 7.2a and the first item in paragraph 7.2b should be pursued on a high priority basis. The remaining remedial measures recommended in Paragraph 7.2b should be accomplished within a reasonable period of time.

d. Necessity for Phase II Inspection

Based upon results of the Phase I inspection, and if the remedial measures recommended in Paragraph 7.2 are undertaken, a Phase II inspection is not felt to be necessary.

7.2 Remedial Measures

a. Alternatives

There are several options that may be considered to reduce the possibility of dam failure or to diminish the harmful consequences of such a failure. Some of these options are:

1. Increase the emergency spillway capacity to pass the PMF, without overtopping the dam. The spillway should also be protected to prevent excessive erosion during the occurrence of the PMF.

2. Increase the height of the dam in order to pass the PMF without overtopping the dam; an investigation should also include studying the effects that increasing the height of the dam would have on the structural stability of the present embankment. The overtopping depth during the occurrence of the PMF, stated in Section 5.1d, is not the required or recommended increase in the height of the dam.

3. A combination of 1 and 2 above.

b. O & M Procedures

1. Further investigation of the three areas of seepage and possible seepage observed along the downstream slope should be undertaken to determine the seriousness of the condition. The investigation should be carried out under the direction of a qualified professional engineer and repairs made as required.

2. The damage to the principal spillway drop inlet sidewall should be repaired.

3. The debris should be removed from the drop inlet structure and measures taken to prevent debris from entering the structure in the future.

4. The undermining of the concrete principal spillway outlet pad should be repaired and the area protected from future damage.

5. The joint displacement in the principal spillway outlet pad should be monitored and corrective repairs should be made when necessary.

6. The voids in the surfacing of the back slope of the emergency spillway should be repaired. This area sould be inspected closely to ensure that future damage is repaired as soon as possible.

7. The minor wave erosion on the upstream slope should be monitored, and, if the erosion continues, protective mesures should be employed to protect the slope from further damage.

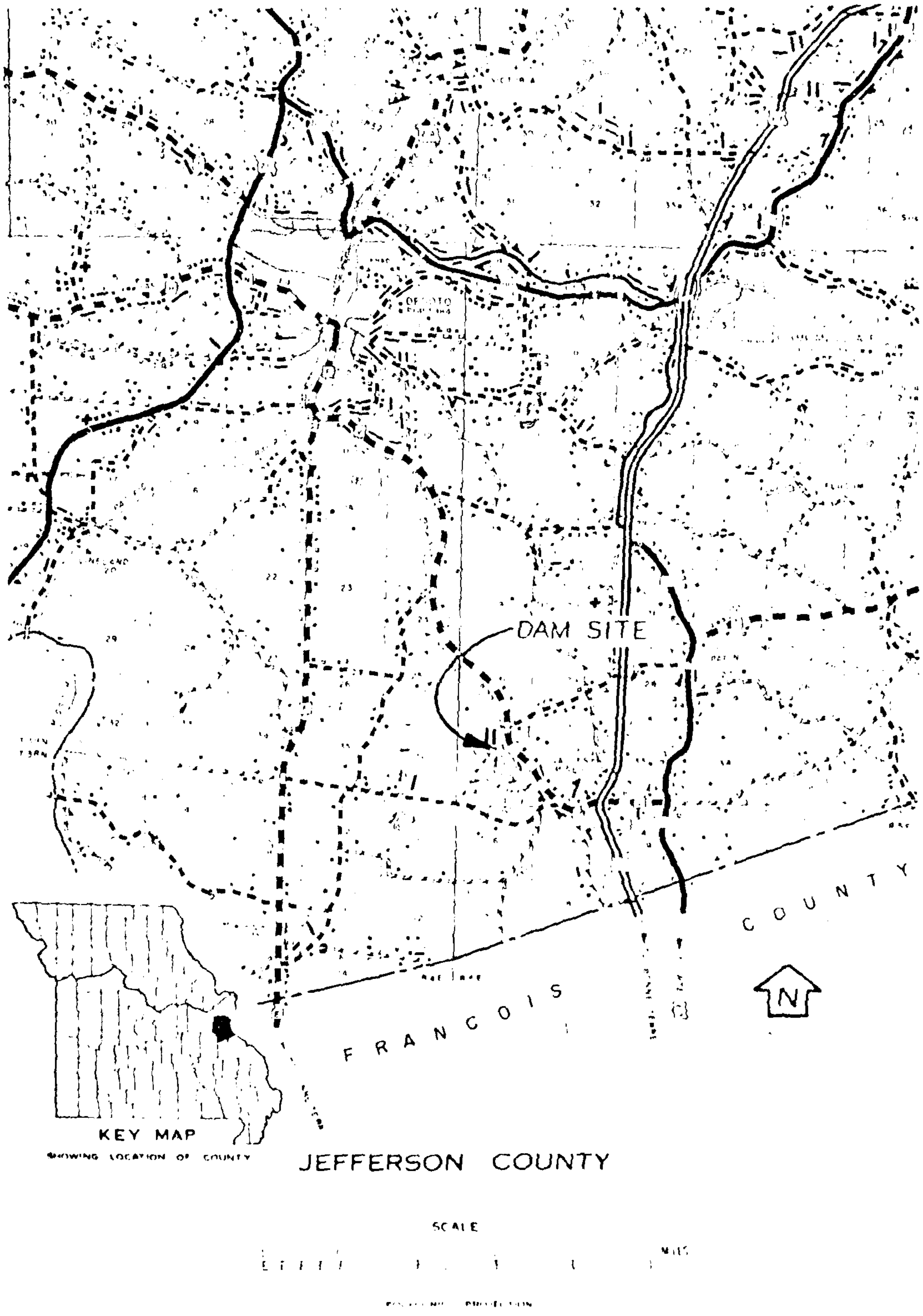
8. Seepage and stability analyses should be performed by a professional engineer experienced in the design and construction of earth dams.

9. The owner should initiate the following programs:

   (a) Periodic inspection of the dam by a professional engineer experienced in the design and construction of earth dams.

   (b) Set up a maintenance schedule and log all repairs and maintenance.

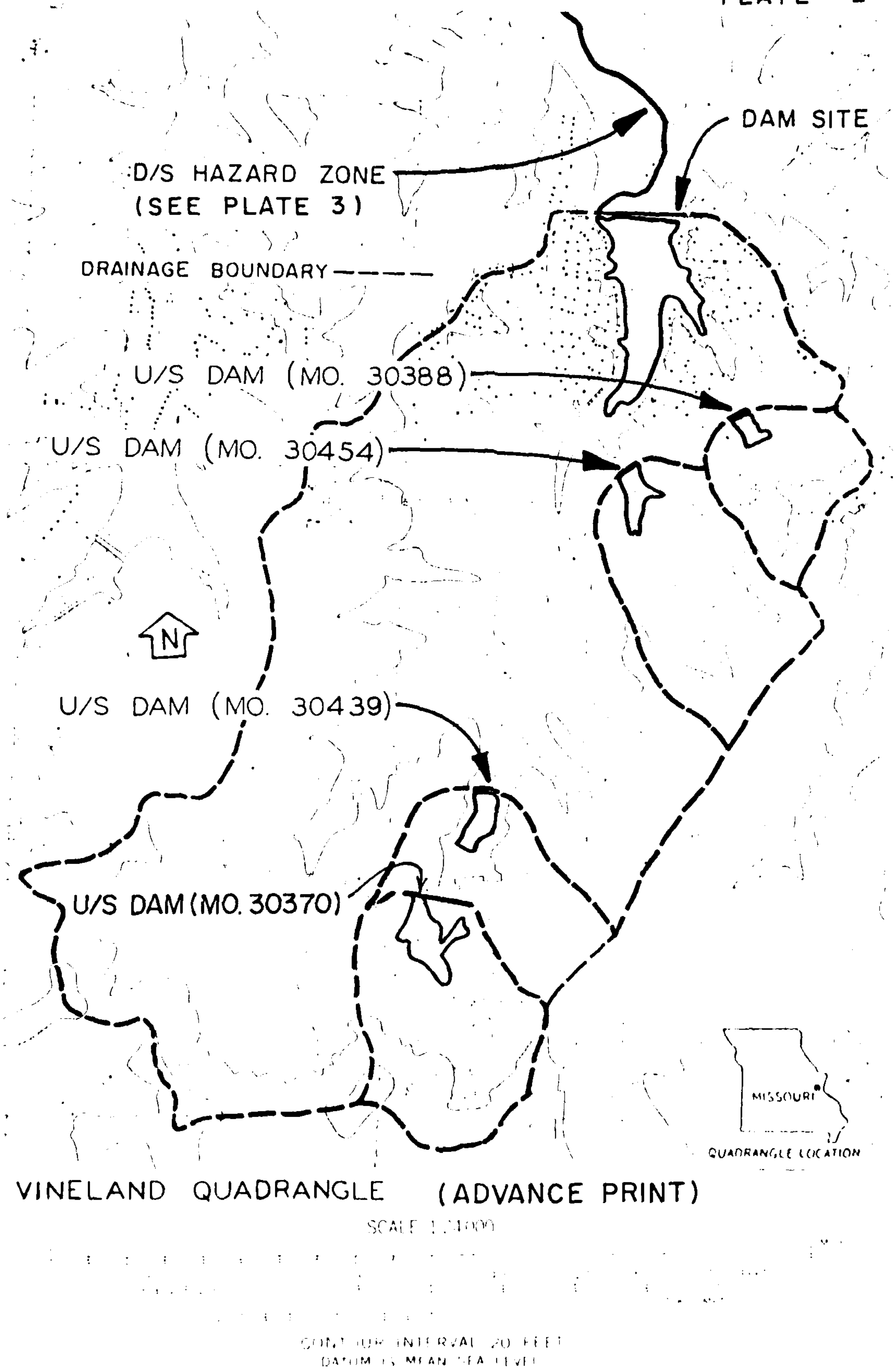
PLATES



LOCATION MAP - VALLE LAKE DAM

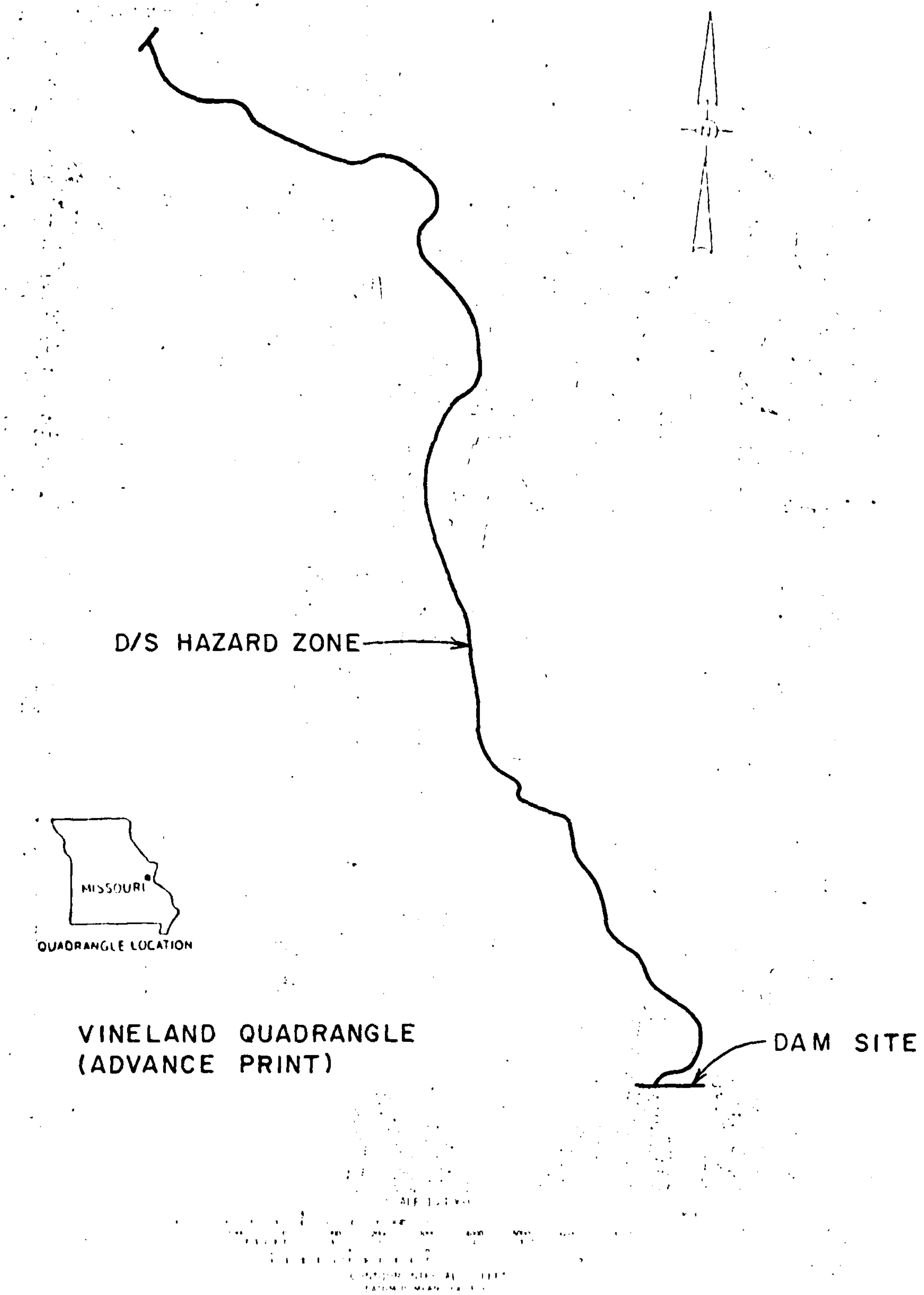
MO. 30438

PLATE 2

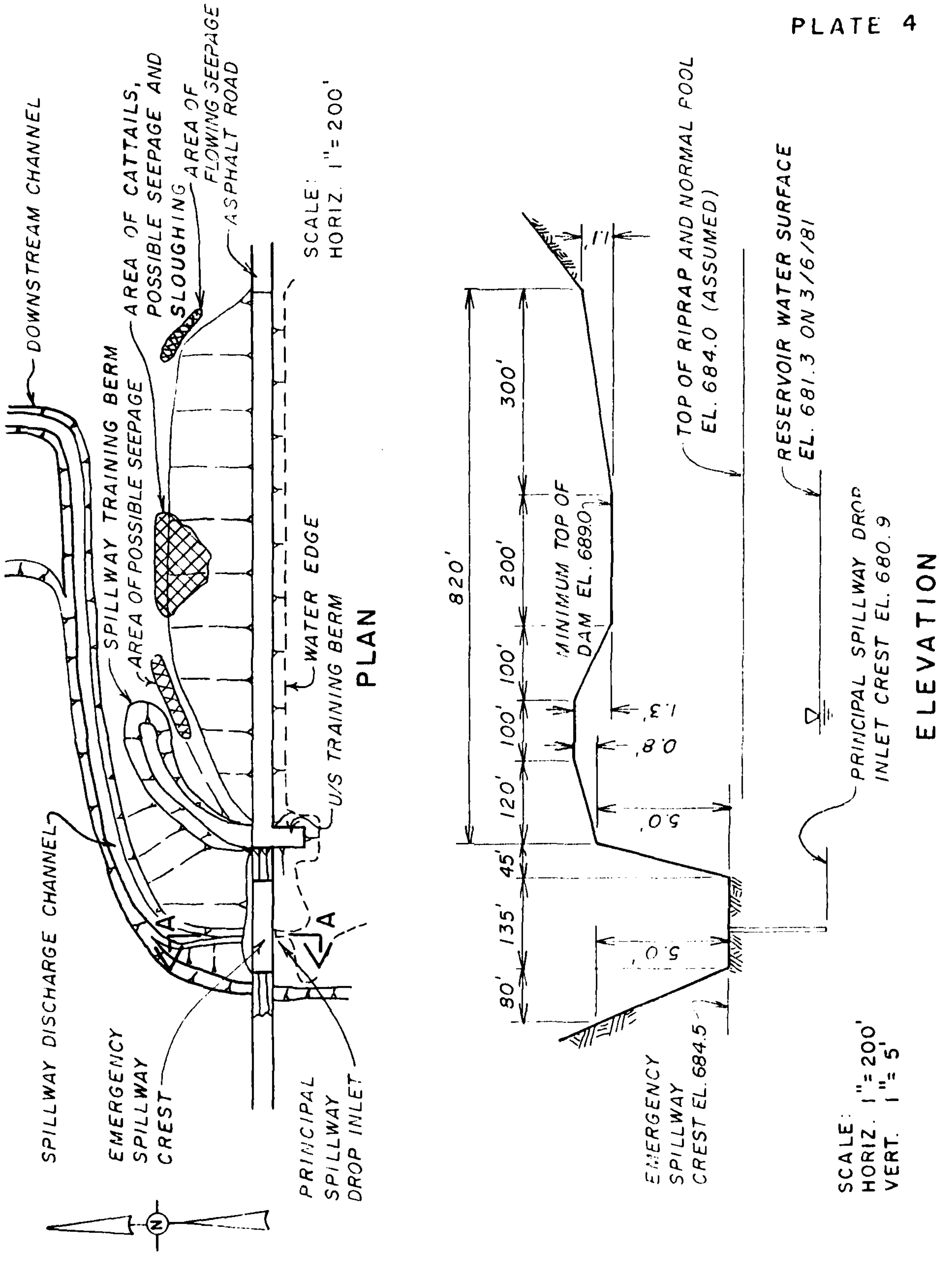


VALLE LAKE DAM (MO. 30438)
DRAINAGE BASIN

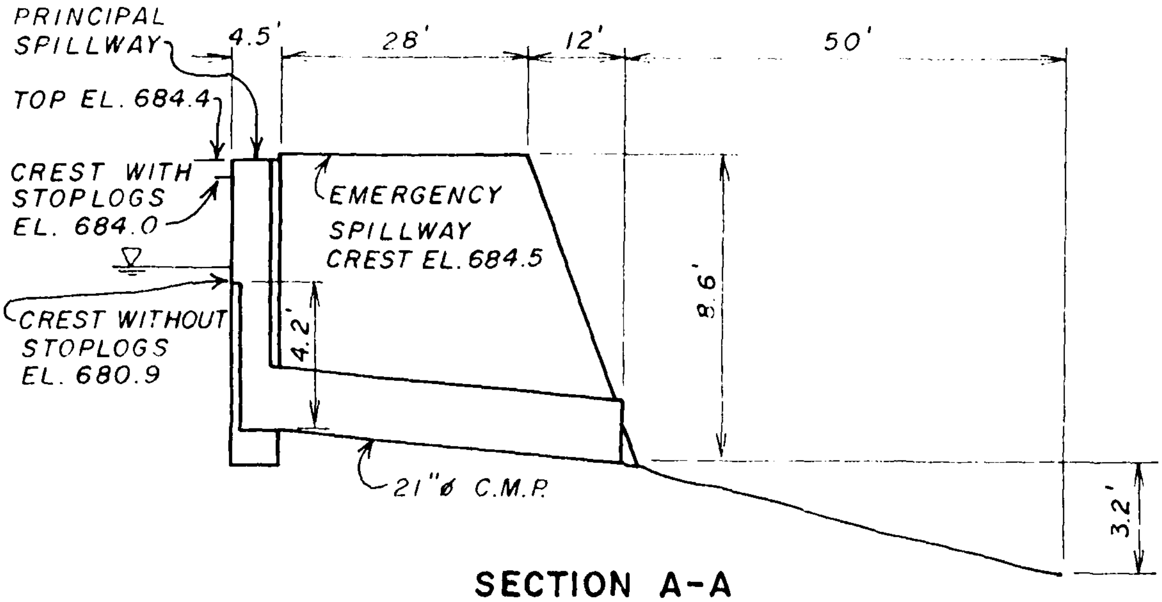
PLATE 3



VALLE LAKE DAM (MO. 30438)
DOWNSTREAM HAZARD ZONE

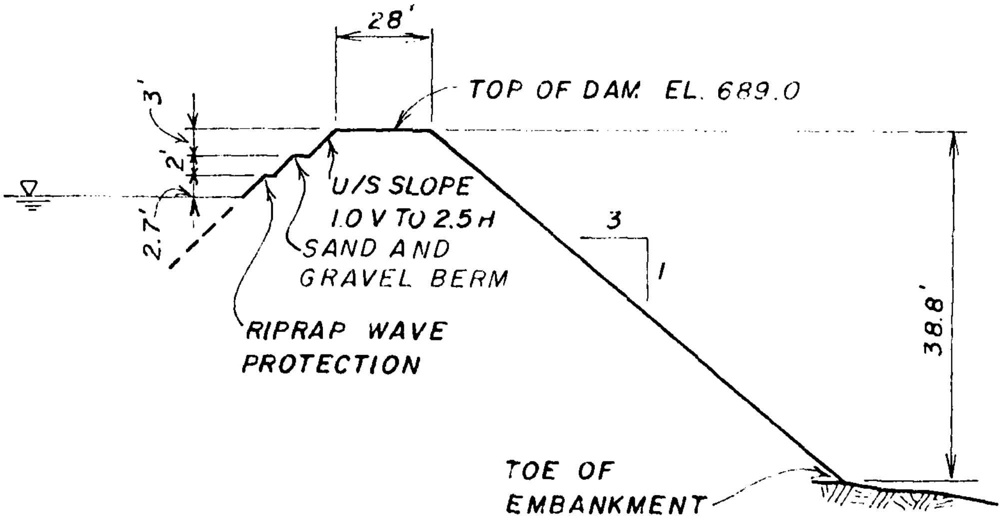


VALLE LAKE DAM (MO. 30438)
PLAN AND ELEVATION
(SHEET 1 OF 2)



SECTION A-A
(SPILLWAY PROFILE)

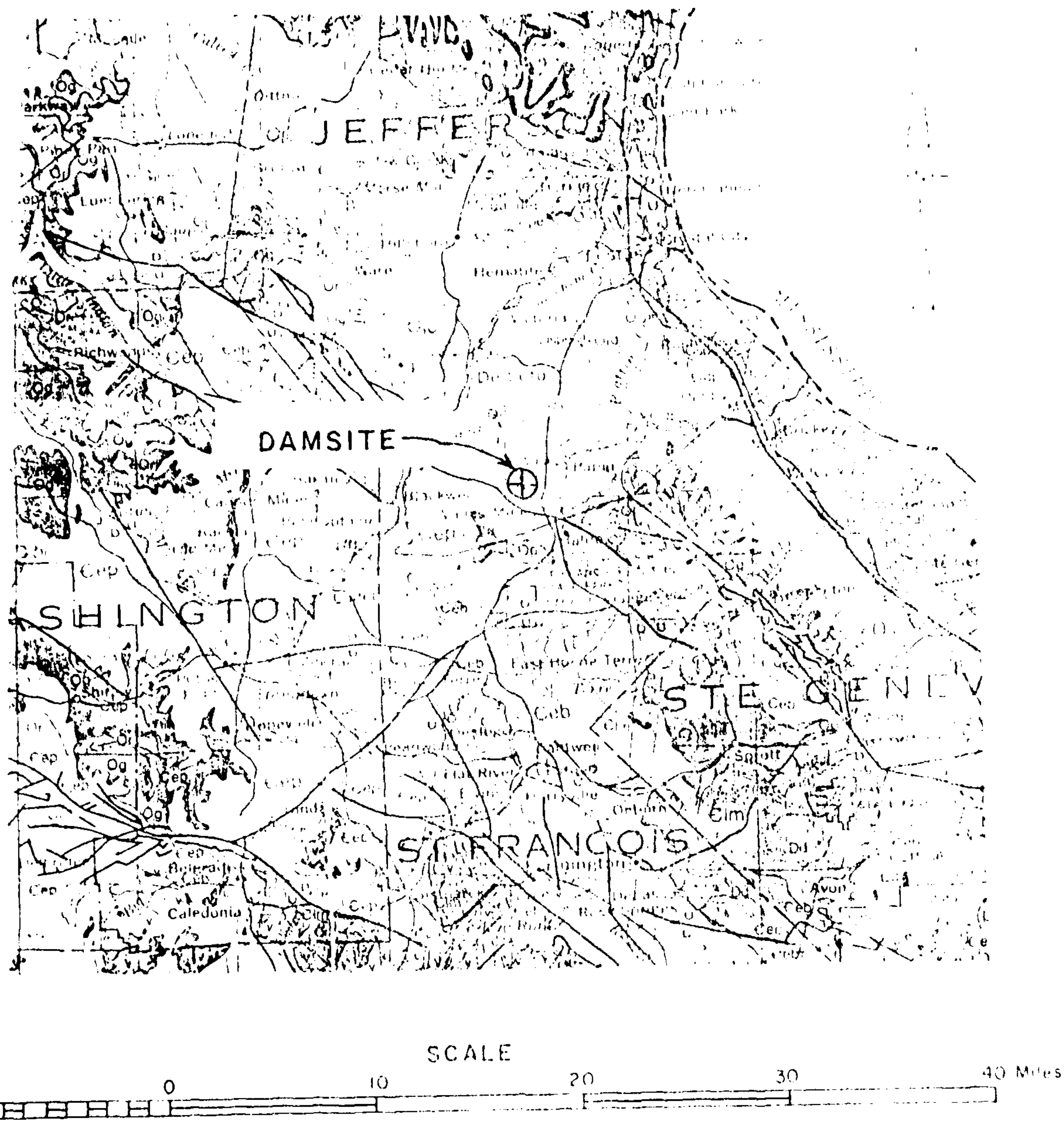
SCALE:
HORIZ. 1" = 20'
VERT. 1" = 5'



MAXIMUM SECTION

SCALE:
HORIZ. 1" = 50'
VERT. 1" = 20'

VALLE LAKE DAM (MO. 30438)
SPILLWAY PROFILE AND MAXIMUM SECTION
(SHEET 2 OF 2)



⊕ LOCATION OF DAM

NOTE: LEGEND FOR THIS MAP IS ON PLATES 7 AND 8.

REFERENCE:

GEOLOGIC MAP OF MISSOURI
DEPARTMENT OF NATURAL RESOURCES
MISSOURI GEOLOGICAL SURVEY
KENNETH H. ANDERSON, 1979

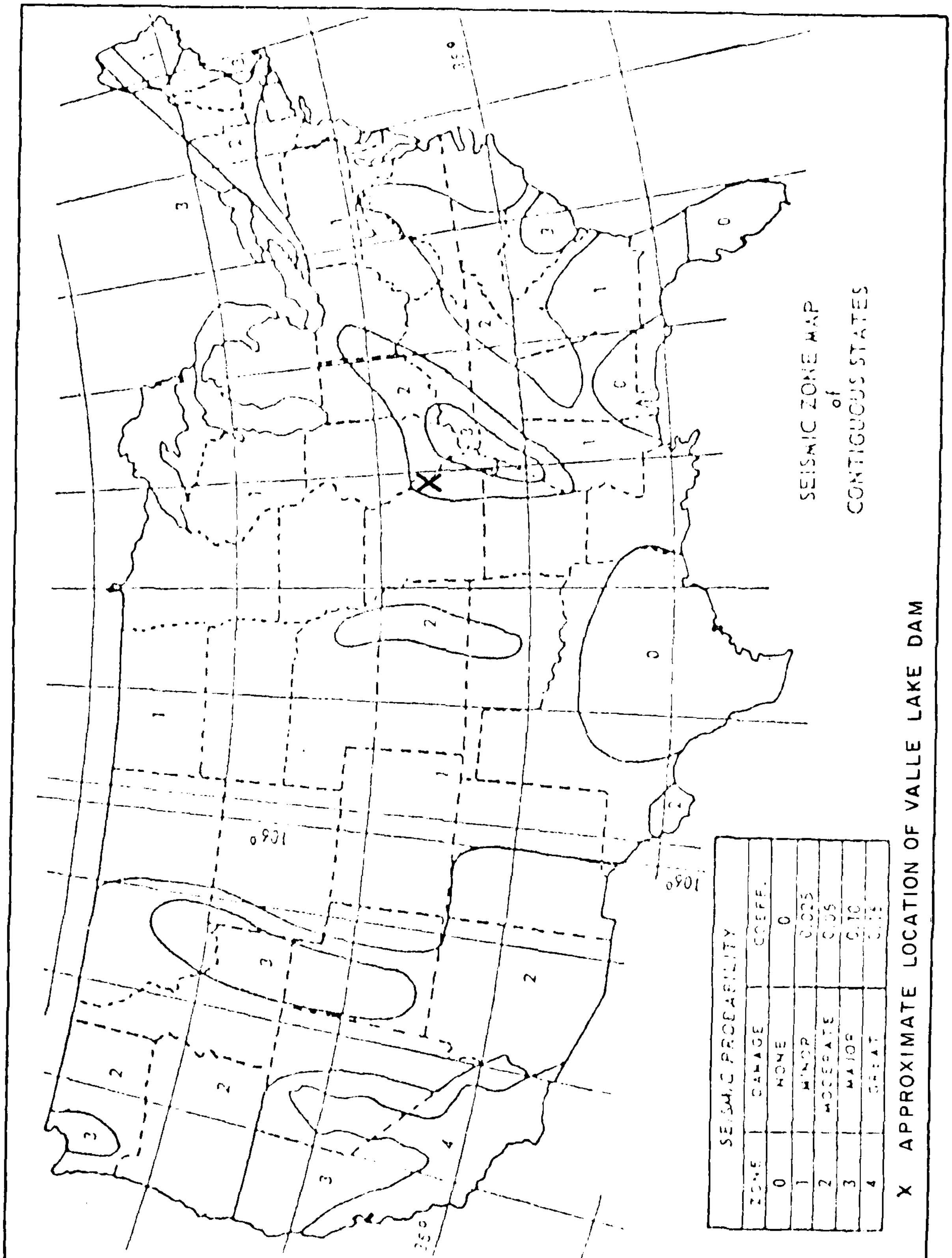
REGIONAL GEOLOGICAL MAP
OF
VALLE LAKE DAM

LEGEND

PERIOD	SYMBOL	DESCRIPTION
QUATERNARY	Qal	ALLUVIUM: SAND, SILT, GRAVEL
MISSISSIPPIAN	Mo	KEOKUK – BURLINGTON FORMATION: CHERTY GRAYISH BROWN SANDY LIMESTONE
	Mk	UNDIFFERENTIATED CHOUTEAU GROUP: LIMESTONE
	Mk	HANNIBAL FORMATION: SHALE AND SILTSTONE
DEVONIAN	Dd	DIATREMES, KIMBERLITES, CARBONATITES
ORDOVICIAN	Omk	MAQUOKETA SHALE, KIMMSWICK LIMESTONE
	Odp	DECORAH FORMATION: GREEN TO GRAY CALCAREOUS SHALE WITH THIN FOSSILIFEROUS LIMESTONE
	Ospe	ST. PETER SANDSTONE, EVERTON FORMATION
	Ojd	JOACHIM DOLOMITE
	Ojc	POWELL DOLOMITE, COTTER DOLOMITE
	Or	ROUBIDOUX FORMATION: INTERBEDS OF CHERTY LIMESTONE AND SANDSTONE
	Og	GASCONADE DOLOMITE

LEGEND

PERIOD	SYMBOL	DESCRIPTION
CAMBRIAN	€ep	EMINENCE DOLOMITE, POTOSI DOLOMITE
	€eb	FRANCONIA AND BONNETERRE FORMATION: INTERBEDDED LIMESTONE, CHERTY LIMESTONE, DOLOMITE AND SILTSTONE
	€lm	LAMOTTE SANDSTONE
PRECAMBRIAN	i	ST. FRANCOIS MOUNTAINS INTRUSIVE
	v	ST. FRANCOIS MOUNTAINS VOLCANIC
	U D	NORMAL FAULT
	U D	INFERRED FAULT
	U =	UPTHROWN SIDE; D = DOWNTHROWN SIDE


SEISMIC ZONE MAP
of
CONTIGUOUS STATES
SEISMIC PROBABILITY
ZONE
DAMAGE
COEFF.
0
NONE
0
1
MINOR
0.025
2
MODERATE
0.05
3
MAJOR
0.10
4
GREAT
X APPROXIMATE LOCATION OF VALLE LAKE DAM

APPENDIX A

PHOTOGRAPHS TAKEN DURING INSPECTION

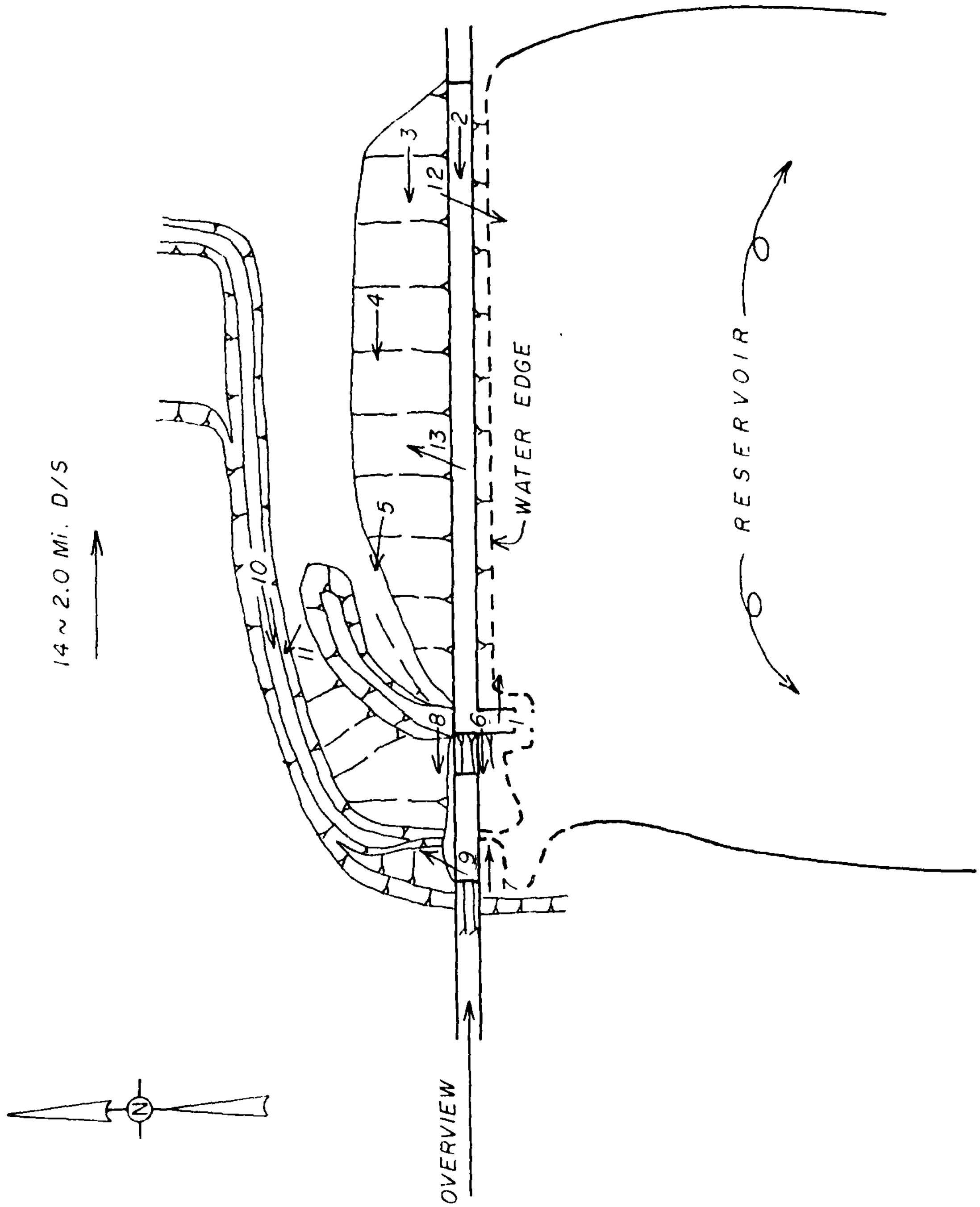


PHOTO INDEX
FOR
VALLE LAKE DAM

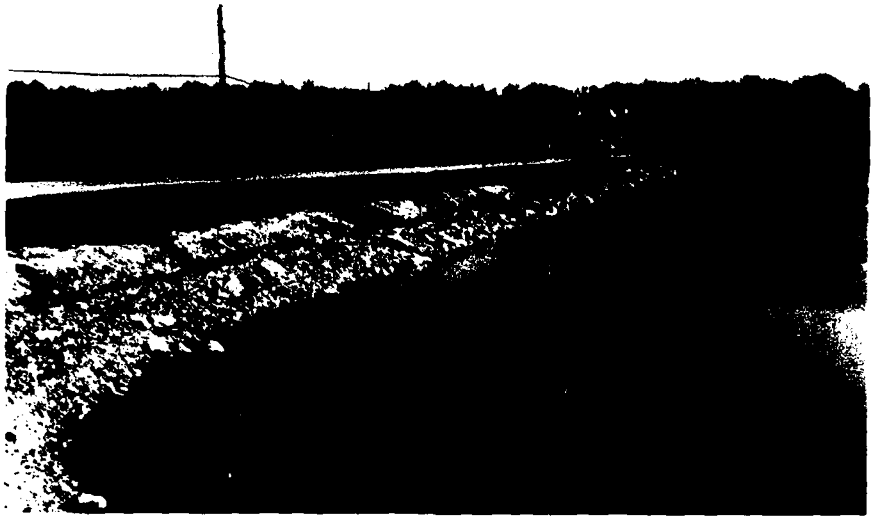


Photo 1 - View of the upstream slope from the left abutment.

Photo 2 - View of the top of dam from the right abutment.

Photo 3 - View of the downstream slope from the right abutment. Note the training berm of the spillway discharge channel in the background.

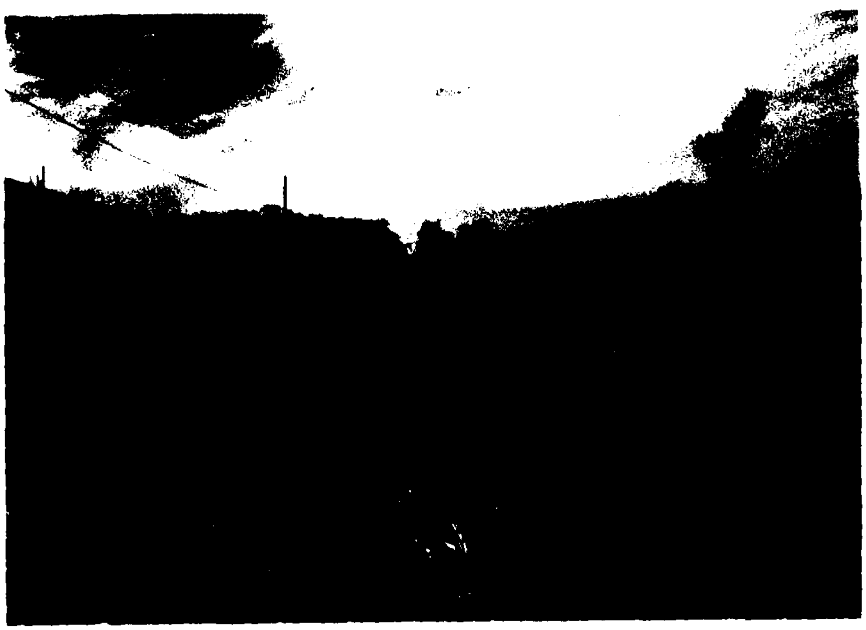


Photo 4 - Close-up view of the area of possible seepage on the downstream slope showing the cattails and scarps.

Photo 5 - View of the area of possible seepage on the left abutment. Note the spillway training berm in the background and the outcropping of bedrock.

Photo 6 - View of the principal spillway drop inlet and emergency spillway broad-crested weir looking toward the left abutment.

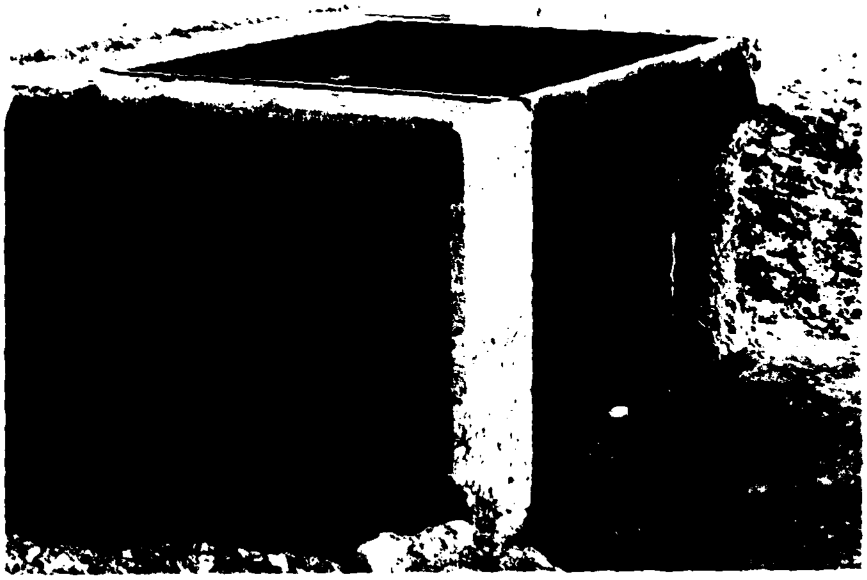


Photo 7 - Close-up view of the principal spillway drop inlet showing the concrete deterioration of the stoplog slot.

Photo 8 - View of concrete and riprap covering of the slope below the emergency spillway. The principal spillway outlet is in the center of the photo.

Photo 9 - View of the spillway discharge channel looking downstream from the emergency spillway crest.

Photo 10 - View of the spillway discharge channel looking upstream. Note the bedrock outcroppings.

Photo 11 - Close-up view of an outcropping of thinly bedded dolomite in the spillway discharge channel.

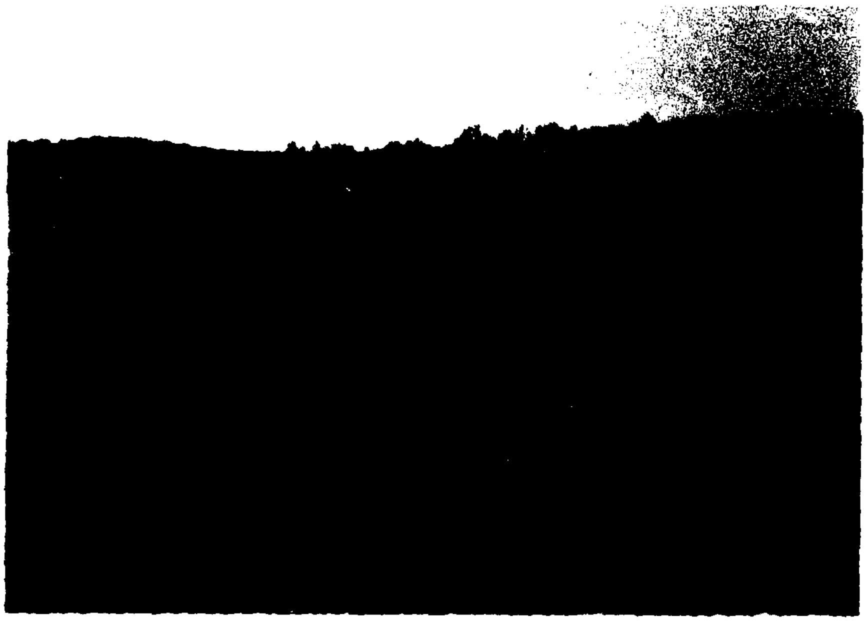


Photo 12 - View of the reservoir and rim.

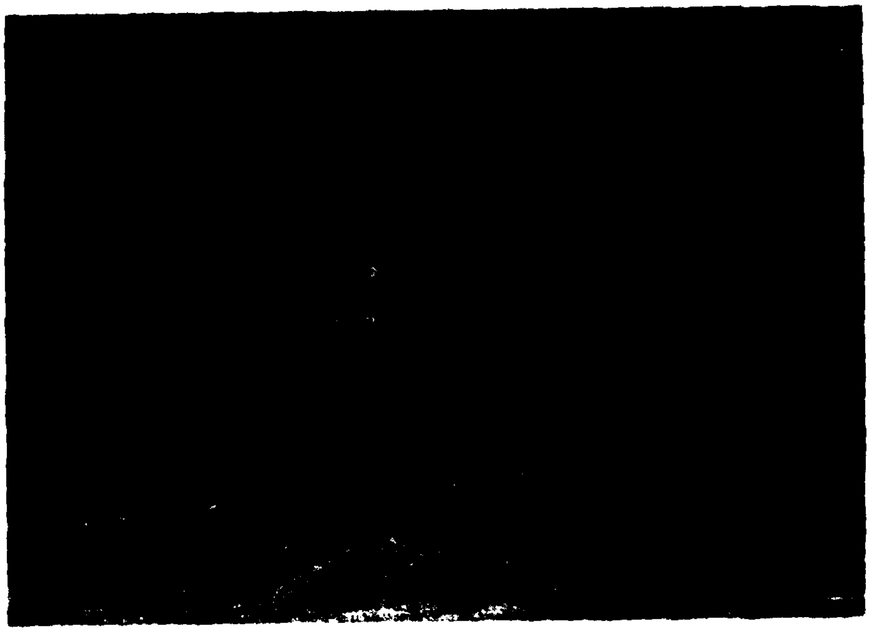


Photo 13 - View of a dwelling in the downstream hazard zone and the downstream channel taken from the top of the dam. The spillway discharge channel is in the foreground.

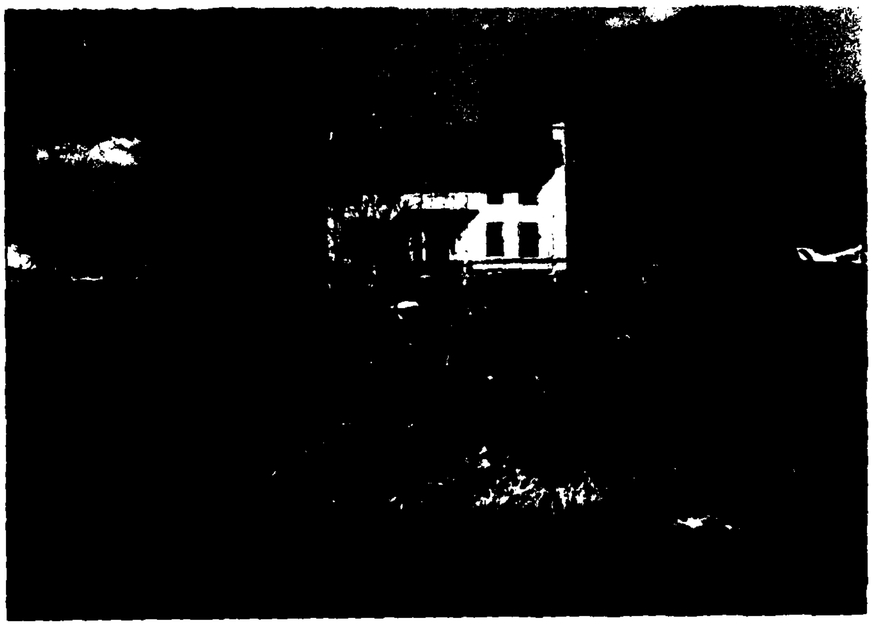


Photo 14 - View of a dwelling in the downstream hazard zone looking across Joachim Creek.

APPENDIX B

HYDROLOGIC AND HYDRAULIC COMPUTATIONS

VALLE LAKE DAM

HYDROLOGIC AND HYDRAULIC DATA, ASSUMPTIONS AND METHODOLOGY

1. SCS Unit Hydrograph procedures and the HEC-1DB computer program are used to develop the inflow hydrographs. The hydrologic inputs are as follows:

   (a) 24-hour Probable Maximum Precipitation from Hydrometeorological Report No. 33, and 24-hour 100-year rainfall and 24-hour 10-year rainfall of Ste. Genevieve, Missouri.

   (b) Drainage area = 2.98 square miles. (including the drainage areas of the U/S dams)

   (c) Lag time = 0.42 hours.

   (d) Hydrologic Soil Group:
   Soil Group "C".

   (e) Runoff curve number:
   CN = 73 for AMC II and CN = 87 for AMC III.

2. Flow rates through the principal spillway are based on calculating discharges for different flow regimes and determining which regime controls. Flow rates through the emergency spillway are based on HEC-2 generated profiles assuming critical depth at the downstream edge of the crest and a Manning's n = 0.015. Flow rates over the dam are based on the broad-crested weir equation $Q = CLH^{3/2}$ and critical depth assumption, in accordance with the procedures used in the HEC-1 computer program.

3. The principal and emergency spillways and the dam overtop rating curves are hand calculated and combined as shown on pages B-5 through B-16. This combined rating curve is input into HEC-1DB on the Y4 and Y5 cards. The $L and $V cards are, therefore, not used.

4. Floods are routed through Valle Lake to determine the capability of the spillways. The analysis of Valle Lake Dam included the hypothetical breach of the four upstream dams for those floods during which the respective dams were overtopped.

FC-14 PRC ENGINEERING CONSULTANTS, INC.

DAM SAFETY INSPECTION / MISSOURI — SHEET NO. ___ OF ___

DAM NAME: Valle Lake Dam (MO. 30438) — JOB NO. 1283

UNIT HYDROGRAPH PARAMETERS — BY JFK — DATE 4/22/81

1) DRAINAGE AREA, $A = 2.16$ sq. mi = (1380. acres) (not including D.A. of U/S Dams)

2) LENGTH OF STREAM, $L = (6.0'' \times 2000' = 12{,}000') = 2.27$ mi.

3) ELEVATION AT DRAINAGE DIVIDE ALONG THE LONGEST STREAM,

$H_1 = 1040$

4) ELEVATION OF RESERVOIR AT SPILLWAY CREST, $H_2 = 684$

5) ELEVATION OF CHANNEL BED AT 0.85L, $E_{85} = 850$

6) ELEVATION OF CHANNEL BED AT 0.10L, $E_{10} = 695$

7) AVERAGE SLOPE OF THE CHANNEL, $S_{AVG} = (E_{85} - E_{10}) / 0.75L = 0.017$

8) TIME OF CONCENTRATION:

A) BY KIRPICH'S EQUATION,

$t_c = [(11.9 \times L^3)/(H_1 - H_2)]^{0.385} = [11.9 \times (2.27)^3 / 356]^{.385} = 0.70$

B) BY VELOCITY ESTIMATE,

SLOPE = 1.7% ⟹ AVG. VELOCITY = 2 fps

$t_c = L/V = 12{,}000' / 2 \text{ fps} \times 1/3600 \text{ s/hr} = 1.67$ hrs.

USE $t_c = 0.70$

9) LAG TIME, $t_\ell = 0.6\, t_c = 0.42$

10) UNIT DURATION, $D \leq t_\ell / 3 = 0.14$ < 0.16 hr.

USE D = 0.083

11) TIME TO PEAK, $T_p = D/2 + t_\ell = 0.46$

12) PEAK DISCHARGE,

$q_p = (484 \times A) / T_p = (484 \times 2.16) / 0.46 = 2{,}270$ cfs

ECI-4 PRC ENGINEERING CONSULTANTS, INC.

Dam Safety Inspection - Missouri SHEET NO. ______ OF ______

Valle Lake Dam (MO. 30438) JOB NO. 1283

Reservoir Elevation-Area Data BY JFK DATE 4/22/81

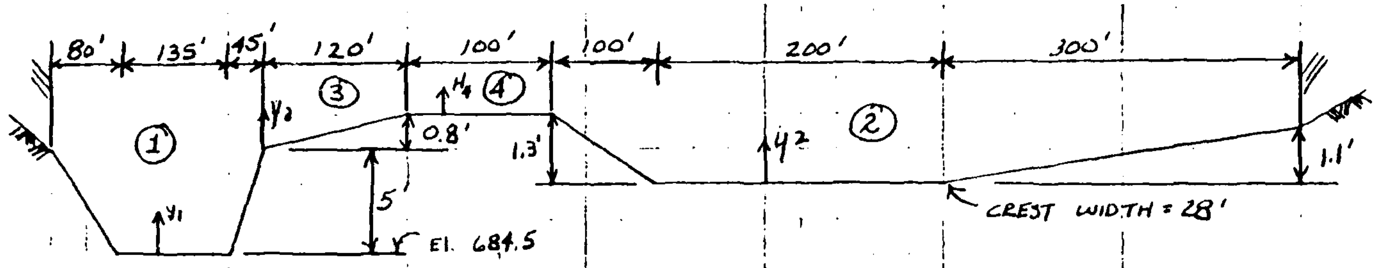
Elevation (ft., MSL)	Area (acres)	Remarks
655	0	Estimated Streambed U/S of Dam
660	2.0	Interpolated
670	12.5	Interpolated
680	32.0	Interpolated
680.9	34.0	Principal Spillway without stoplogs
684	42.5	Principal Spillway Weir Crest (assumed)
684.5	44.0	Emergency Spillway Crest
689	57.5	Minimum Top of Dam
700	94.0	Measured from Vineland 7.5' USGS Quad
710	131.5	Interpolated
720	171.0	Measured from Vineland 7.5' USGS Quad

ECI-4 PRC ENGINEERING CONSULTANTS, INC.

Dam Safety Inspection - Missouri — SHEET NO. ____ OF ____

Valle Lake Dam (MO 30438) — JOB NO. 1283

Spillway and Overtop Rating Curve — BY JFK — DATE 4/27/81



Section 1: for $0 < y_1 < 5.0$ — for $y_1 \geq 5.0$, $W.S.EL. = 684.5 + y_1 + V_1^2/2g$

$A_1 = 135 y_1 + 12.5 y_1^2$ — $A_1 = 260. y_1 - 312.5$

$T_1 = 135. + 25 y_1$ — $T_1 = 260.$

HEC-2 step backwater program used to generate emergency spillway rating curve

Section 2*: for $0 < y_2 < 1.1$ — for $1.1 \leq y_2 < 1.3$ — for $y_2 \geq 1.3$

$A_2 = 200 y_2 + 174.8 y_2^2$ — $A_2 = 500 y_2 + 38.5 y_2^2 - 165.$ — $A_2 = 600 y_2 - 230.$

$T_2 = 200. + 349.7 y_2$ — $T_2 = 500. + 76.9 y_2$ — $T_2 = 600.$

$y_2 = H_2 - V_2^2/2g$

Section 3*: for $0 < y_3 \leq 0.8$ — for $y_3 > 0.8$; $H_3 = W.S.EL - 689.5$

$y_3 = 4/5 H_3$ — $y_3 = 2/3 (H_3 + 0.2)$

$A_3 = 75. y_3^2$ — $A_3 = 120. y_3 - 48.$

$T_3 = 150. y_3$ — $T_3 = 120.$

Section 4: for all H_4 , $H_4 = W.S.EL - 690.3$

$Q_4 = C_4 L H_4^{1.5}$

* $Q = \sqrt{A^3 g / T}$

ECI-4 **PRC ENGINEERING CONSULTANTS, INC.**

Dam Safety Inspection · Missouri — SHEET NO. ____ OF ____
Valle Lake Dam (MO 30438) — JOB NO. 1283
Spillway and Overtop Rating Curve — BY JFK — DATE 4/27/81

← Refer to HEC-2 Printout →

Y_1	A_1	T_1	V_1	Q_1	W.S.EL.	H_2	Y_2	A_2	T_2	Q_2	H_3	Y_3	A_3	T_3	Q_3	H_4	C_4	Q_4
0	0	0	0	0	684.5													
0.3	37.1	141.7	1.4	50	684.8													
0.4	55.5	144.9	1.8	100	684.9													
0.7	93.3	151.3	2.7	250	685.3													
1.0	141.1	159.0	3.5	500	685.7													
1.4	216.1	170.4	4.6	1000	686.3													
2.1	342.0	188.0	5.9	2000	687.2													
2.7	455.9	202.5	6.6	3000	687.9													
3.2	553.7	214.3	7.2	4000	688.5													
3.6	647.2	224.9	7.7	5000	689.0	0	0	0	0	0	0	0	0	0	0			
4.3	821.5	243.5	8.5	7000	690.0	1.0	0.71	230.1	448.3	935.7	0.5	0.4	12.0	60.0	30.5	0	0	0
4.7	904.1	251.9	8.9	8000	690.4	1.4	1.00	374.8	549.7	1756.4	0.9	0.7	38.9	108.0	132.4	0.1	2.93	9.3
5.3	1057.7	260	9.5	10000	691.2	2.2	1.60	730.0	600.0	4569.2	1.7	1.27	104.	120.0	549.4	0.9	3.03	258.7
5.8	1192.5	260	10.1	12000	691.9	2.9	2.05	1000.0	600.0	7325.8	2.4	1.73	160.	120.0	1048.4	1.6	3.04	615.3
6.3	1324.2	260.	10.6	14000	692.5	3.5	2.46	1246.0	600.0	10188.9	3.0	2.13	208.	120.0	1553.9	2.2	3.04	991.9

W.S.EL.	Q_{TOTAL}
684.5	0
684.8	50
684.9	100
685.3	250
685.7	500
686.3	1000
687.2	2000
687.9	3000
688.5	4000
689.0	5000
690.0	7966
690.4	9898
691.2	15377
691.9	20990
692.5	26735

HEC-2 INPUT AND SUMMARY TABLE

```
**************************************************
 HEC2 RELEASE DATED NOV 76 UPDATED APRIL 1980
 ERROR CORR -  01,02,03,04
 MODIFICATION -  50,51,52,53,54
**************************************************

T1        MISSOURI DAM SAFETY
T2        EMERGENCY SPILLWAY RATING CURVE
T3        VALLE LAKE DAM (MO.30436)

J1  ICHECK    INQ       NINV      IDIR      STRT      METRIC    HVINS     Q         WSEL      FQ

     -1.       0.        0.        1.  -1.000000   0.00      0.0       0.      678.000    0.000

J2  NPROF     IPLOT     PREVS     XSECV     XSECH     FN        ALLDC     IBW       CHNIM     ITRACE

    1.000     0.000    -1.000     0.000     0.000     0.000     1.000     0.000     0.000     0.000

J3  VARIABLE CODES FOR SUMMARY PRINTOUT

   38.000     4.000    25.000     4.000    26.000    10.000    43.000     3.000     1.000    17.000

J5  LPRNT     NUMSEC                  *********REQUESTED SECTION NUMBERS*********

  -10.000   -10.000     0.000     0.033     0.200     0.000     0.000     0.000     0.000     0.000
J6  IHLEQ     ICOPY

    1.000     0.000     0.000     0.000     0.000     0.000     0.000     0.000     0.000     0.000

NC      .033      .030      .050      .100      .300     0.000     0.000     0.000     0.000     0.000
WT    14.000    50.000   170.000   251.700   500.000  1000.000  2000.000  3000.000  4000.000  5000.000
QT  7000.000  8000.000 10000.000 12000.000 14000.000     0.000     0.000     0.000     0.000     0.000

X1     1.000     4.000     0.000   260.000     0.000     0.000     0.000     0.000   -11.600     0.000
GR   689.500     0.000   684.500    80.000   684.500   215.000   689.500   260.000     0.000     0.000

X1     2.000     0.000     0.000     0.000    50.000    50.000    50.000     0.000     3.200     0.000
NC      .015      .015      .015      .100      .300     0.000     0.000     0.000     0.000     0.000

X1     3.000     0.000     0.000     0.000    12.000    12.000    12.000     0.000     8.600     0.000

X1     4.000     0.000     0.000     0.000    28.000    28.000    28.000     0.000     0.000     0.000
EJ     0.000     0.000     0.000     0.000     0.000     0.000     0.000     0.000     0.000     0.000

IHLEQ = 1.  THEREFORE FRICTION LOSS (HL) IS CALCULATED AS A FUNCTION OF
PROFILE TYPE, WHICH CAN VARY FROM REACH TO REACH.  SEE DOCUMENTATION FOR
DETAILS.
```

****
HEC2 RELEASE DATED NOV 76 UPDATED APRI 1980
ERROR CORR - 01,02,03,04
MODIFICATION - 50,51,52,53,54
****

NOTE- ASTERISK (*) AT LEFT OF CROSS-SECTION NUMBER INDICATES MESSAGE IN SUMMARY OF ERRORS LIST

VALLE LAKE DAM (MO.*043)

SUMMARY PRINTOUT

	SECNO	DEPTH	AREA	TOPWID	VCH	HV	Q	EG	10K*S	K*XNCH
*	1.000	.16	22.07	139.03	2.27	.08	50.00	672.94	243.43	30.00
*	1.000	.25	35.15	141.36	2.85	.13	100.00	673.08	211.11	30.00
*	1.000	.47	65.53	146.63	3.82	.23	250.00	673.35	173.70	30.00
*	1.000	.73	105.71	153.33	4.73	.35	500.00	673.78	149.80	30.00
*	1.000	1.15	171.50	163.71	5.83	.53	1000.00	674.38	130.34	30.00
*	1.000	1.78	280.60	179.60	7.13	.79	2000.00	675.27	114.34	30.00
*	1.000	2.33	376.50	192.44	7.97	.99	3000.00	675.99	105.92	30.00
*	1.000	2.74	463.94	203.52	8.62	1.15	4000.00	676.60	101.14	30.00
*	1.000	3.14	547.07	213.49	9.14	1.30	5000.00	677.14	97.25	30.00
*	1.000	3.65	705.21	231.27	9.93	1.53	7000.00	678.08	90.98	30.00
*	1.000	4.13	776.60	238.86	10.30	1.65	8000.00	678.50	89.97	30.00
*	1.000	4.74	921.00	253.52	10.86	1.83	10000.00	679.27	86.22	30.00
*	1.000	5.21	1041.17	260.00	11.53	2.06	12000.00	679.97	85.50	30.00
*	1.000	5.56	1158.74	260.00	12.08	2.27	14000.00	680.63	81.84	30.00
*	2.000	.16	22.03	139.02	2.27	.08	50.00	676.14	244.89	30.00
*	2.000	.25	35.18	141.37	2.84	.13	100.00	676.28	210.39	30.00
*	2.000	.47	65.66	146.65	3.41	.23	250.00	676.59	173.05	30.00
*	2.000	.73	105.67	153.32	4.73	.35	500.00	676.98	149.99	30.00
*	2.000	1.15	171.43	163.70	5.83	.53	1000.00	677.58	130.52	30.00
*	2.000	1.78	280.68	179.61	7.13	.79	2000.00	678.47	114.24	30.00
*	2.000	2.30	376.28	192.46	7.97	.99	3000.00	679.19	106.11	30.00
*	2.000	2.74	464.58	203.63	8.61	1.15	4000.00	679.80	100.73	30.00
*	2.000	3.14	547.70	213.57	9.13	1.29	5000.00	680.34	96.92	30.00
*	2.000	3.59	703.10	231.04	9.96	1.54	7000.00	681.28	91.78	30.00
*	2.000	4.16	777.61	238.49	10.29	1.64	8000.00	681.73	89.57	30.00
*	2.000	4.74	921.31	253.50	10.85	1.83	10000.00	682.47	86.14	30.00
*	2.000	5.23	1048.21	260.00	11.45	2.04	12000.00	683.17	85.62	30.00
*	2.000	5.67	1169.73	260.00	12.06	2.26	14000.00	683.83	81.38	30.00

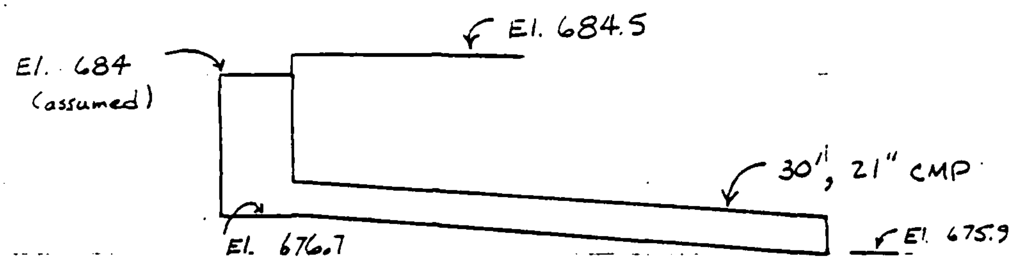
	SLOPE	DEPTH	AREA	TOP WID	VCH	HV	Q	EG	10K*S	K*XNCH
•	3.000	.16	22.34	139.02	2.27	.08	50.00	684.74	61.14	15.00
•	3.000	.29	35.18	141.36	2.84	.13	100.00	684.88	52.63	15.00
•	3.000	.47	65.59	146.64	3.81	.23	250.00	685.19	43.29	15.00
•	3.000	.73	105.66	153.35	4.73	.35	500.00	685.56	37.49	15.00
•	3.000	1.15	171.44	163.76	5.83	.53	1000.00	686.18	32.62	15.00
•	3.000	1.75	286.66	179.61	7.13	.79	2000.00	687.07	28.57	15.00
•	3.000	2.30	376.33	192.46	7.97	.99	3000.00	687.79	26.52	15.00
•	3.000	2.74	464.45	203.59	8.61	1.15	4000.00	688.40	25.20	15.00
•	3.000	3.14	547.57	213.55	9.13	1.29	5000.00	688.94	24.25	15.00
•	3.000	3.64	703.56	231.09	9.95	1.54	7000.00	689.88	22.96	15.00
•	3.000	4.16	777.55	239.96	10.29	1.64	8000.00	690.30	22.41	15.00
•	3.000	4.74	921.28	253.55	10.85	1.83	10000.00	691.07	21.54	15.00
•	3.000	5.23	1040.96	260.00	11.47	2.04	12000.00	691.77	21.02	15.00
•	3.000	5.66	1165.23	260.00	12.07	2.26	14000.00	692.43	20.37	15.00
	4.000	.27	37.65	141.69	1.35	.03	50.00	684.83	11.10	15.00
	4.000	.31	55.49	144.91	1.40	.05	100.00	684.94	11.91	15.00
	4.000	[illegible]	95.34	151.30	2.68	.11	250.00	685.26	13.93	15.00
	4.000	[illegible]	141.09	153.99	3.54	.20	500.00	685.66	15.02	15.00
	4.000	1.42	216.13	170.39	4.63	.33	1000.00	686.26	15.90	15.00
	4.000	2.12	342.04	187.95	5.85	.53	2000.00	687.16	15.70	15.00
	4.000	2.75	455.88	202.53	6.59	.67	3000.00	687.17	14.98	15.00
	4.000	3.17	553.71	214.27	7.22	.81	4000.00	688.48	15.02	15.00
	4.000	3.[illegible]	647.20	224.91	7.73	.93	5000.00	689.03	14.88	15.00
	4.000	4.34	821.53	243.52	8.52	1.13	7000.00	689.97	14.65	15.00
	4.000	4.63	904.10	251.45	8.85	1.22	8000.00	690.39	14.54	15.00
	4.000	5.24	1057.66	260.00	9.45	1.35	10000.00	691.16	14.10	15.00
	4.000	5.7[illegible]	1192.48	260.00	10.26	1.57	12000.00	691.86	13.68	15.00
	4.000	6.2[illegible]	1324.16	260.00	10.57	1.74	14000.00	692.52	13.20	15.00

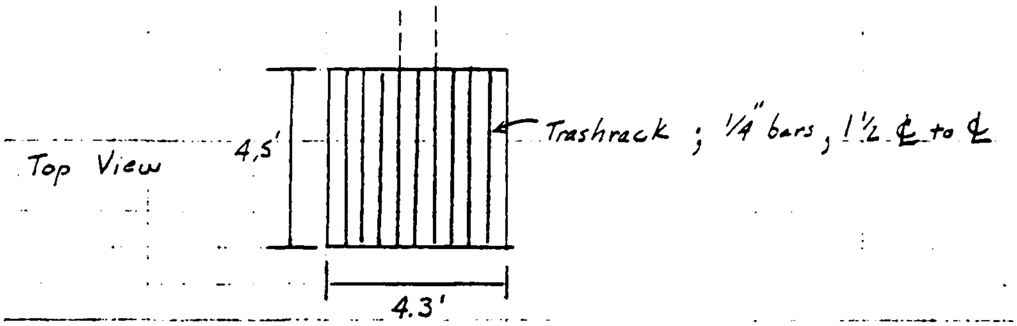
ECI-4 PRC ENGINEERING CONSULTANTS, INC.

Dam Safety Inspection – Missouri — SHEET NO. ____ OF ____

Valle Lake Dam (MO. 30438) — JOB NO. 1283

Principal Spillway Rating Curve — BY JFK — DATE 4/27/81





Weir Flow:

$Q = CLH^{1.5}$, where $L = 13.3$
H = W.S.EL. − 684.0 (to simplify, assume that the concrete sidewalls of the drop structure are at the same elevation as the stoplogs)
$C = 2.90$

W.S.EL.	H	Q
684.0	0	0
684.5	0.5	13.6
685.0	1.0	38.6
685.5	1.5	70.9

ECI-4 PRC ENGINEERING CONSULTANTS, INC.

Dam Safety Inspection - Missouri — SHEET NO. ____ OF ____
Valle Lake Dam (MO. 30438) — JOB NO. 1283
Principal Spillway Rating Curve — BY JFK DATE 4/23/81

Orifice Flow:

$$Q = CA\sqrt{2gH_o} \quad , \quad H_o = W.S.EL. - 677.6$$

$$Q = 0.6\,(\pi\,(.875)^2)\,\sqrt{2gH_o}$$

$$Q = 11.6\,\sqrt{H_o}$$

W.S. EL.	H_o	Q
684.5	6.9	30.5
684.9	7.3	31.3
685.3	7.7	32.2
685.7	8.1	33.0
686.3	8.7	34.2
687.2	9.6	35.9
687.9	10.3	32.2

Pressure Flow:

$H_T = \Sigma K \; V^2/2g$

$V = \sqrt{2g/\Sigma K \; H_T}$

$Q = VA$

$Q = \sqrt{2g/\Sigma K} \; (\pi(0.875)^2) \sqrt{H_T}$, H_T = W.S.EL - T.W.EL.

$K_{entrance} = 0.5$ (at 21" CMP)

$K_{exit} = 1.0$

$K_{friction} = 29.16 \frac{n^2 L}{R_h^{4/3}}$, $n = 0.024$, $L = 30'$, $R_h = \frac{\pi r^2}{2\pi r} = \frac{r}{2} = \frac{.875}{2}$

$= 1.5$ (neglect friction loss in drop structure

$K_{bend} = 0.2$

$K_{trashrack} = 1.45 - 0.45 \left(\frac{a_n}{a_g}\right) - \left(\frac{a_n}{a_g}\right)^2$, where $a_n/a_g = \frac{1\frac{1}{4}}{1\frac{1}{2}} = 5/6$

$= 0.4$

$\Sigma K = 3.6$

$Q = \sqrt{2g/3.6} \; (\pi(0.875)^2) \sqrt{H_T}$

$Q = 10.2 \sqrt{H_T}$

ECI-4 **PRC ENGINEERING CONSULTANTS, INC.**

Dam Safety Inspection - Missouri — SHEET NO. ____ OF ____

Valle Lake Dam (MO. 30438) — JOB NO. 1283

Principal Spillway Rating Curve — BY JFK DATE 4/23/81

W.S.EL.	T.W.EL.	H_T	Q
684.5	676.8	7.7	28.3
684.8	676.8	8.0	28.9
684.9	676.8	8.1	29.0
685.3	676.8	8.5	29.7
685.7	677.0	8.7	30.1
686.3	677.6	8.7	30.1
687.2	678.5	8.7	30.1
687.9	679.2	8.7	30.1
688.5	679.8	8.7	30.1
689.0	680.3	8.7	30.1
690.0	681.3	8.7	30.1
690.4	681.7	8.7	30.1
691.2	682.5	8.7	30.1
691.9	683.2	8.7	30.1
692.5	683.8	8.7	30.1

ECI-4 **PRC ENGINEERING CONSULTANTS, INC.**

Dam Safety Inspection - Missouri SHEET NO. ______ OF ______

Valle Lake Dam (MO 30438) JOB NO. 1283

Principal Spillway Rating Curve BY JFK DATE 4/23/81

W.S.EL.	Q	Flow Regime
684	0	
684.5	13.6	Weir Flow
684.8	28.9	
684.9	29.0	Pressure Flow
685.3	29.7	" "
685.7	30.1	" "
686.3	30.1	" "
687.2	30.1	" "
687.9	30.1	" "
688.5	30.1	" "
689.0	30.1	" "
690.0	30.1	" "
690.4	30.1	" "
691.2	30.1	" "
691.9	30.1	" "
692.5	30.1	" "

ECI-4 PRC ENGINEERING CONSULTANTS, INC.

Dam Safety Inspection - Missouri SHEET NO. ____ OF ____

Valle Lake Dam (MO 30438) JOB NO. 1283

Combined Rating Curve BY JFK DATE 4/27/81

W.S.EL.	$Q_{P.\ SPLWY}$	$Q_{SRWY\ \&\ O.T.}$	Q_{TOTAL}
684	0		0
684.5	14	0	14
684.8	29	50	79
684.9	29	100	129
685.3	30	250	280
685.7	30	500	530
686.3	30	1000	1030
687.2	30	2000	2030
687.9	30	3000	3030
688.5	30	4000	4030
689.0	30	5000	5030
690.0	30	7966	7996
690.4	30	9898	9928
691.2	30	15377	15407
691.9	30	20990	21020
692.5	30	26735	26765

SUMMARY OF PMF AND ONE-HALF PMF ROUTING

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**********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION       JULY 1978
  LAST MODIFICATION  01 APR 80
**********************************
A1        MISSOURI DAM SAFETY
A2        VALLE LAKE DAM (MO.30438)
A3        PMF AND 60 PERCENT PMF
B      300       0       5       0       0       0       0       0       0       0
B1       5
J        1       2       1
J1       1      .5
K              UVM                                       1
K1        RUNOFF CALCULATION FOR UPPER VALLE MINES DAM
M        1       2     .25             .25       1                       1
P               26     100     120     130
T                                                       -1     -87
W2             .12
X                        1
K        1     UVM                                       1
K1        ROUTE HYDROGRAPH THROUGH UPPER VALLE MINES DAM
Y                                1       1
Y1       1                                          -827.5      -1
Y4   827.5   827.8   828.1   828.5   828.8   829.4
Y5       0      45     371    1118    1881    3924
$A       0     6.5      11      16      16    20.5    39.6
$E     813     820   823.7   827.5   827.7     830     840
$$   827.5
$D   827.7
$B      19      .5     815       1   827.5   827.7
K              LVM                                       1
K1        RUNOFF CALCULATION FOR LOWER VALLE MINES DAM
M        1       2     .19             .19       1                       1
P               26     100     120     130
T                                                       -1     -87
W2             .11
X                        1
K        2     LVM                                       1
K1        COMBINE ROUTED HYDROGRAPH AND RUNOFF HYDROGRAPH
```

```
K        1    LVM                                            1
K1           ROUTE COMBINED HYDROGRAPH THROUGH LOWER VALLE LAKE DAM
Y                                 1         1
Y1       1                                               -785.2      -1
Y4   782.5   783.4     784    784.5       785    785.5     786
Y5       0      30     120      295       520      907    1573
$A       0       3     6.5        7         8       12      20
$E     765     775   782.5    783.4     785.2      790     800
$$   782.5
$D   785.2
$B      10      .5     770        1     782.5    785.2
K        1    LVM                                            1
K1           STREAM ROUTING OF HYDROGRAPH TO VALLE LAKE DAM
Y                                 1         1
Y1       1
Y6     .08    .045     .08      684       726     6000   .0013
```

B-19

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Y7       0     726     200      706       320      686     340     684     380     684
Y7     400     686     530      706       620      726
K              LL                                            1
K1           RUNOFF CALCULATION FOR LUCAS LAKE DAM
M        1       2     .25                .25        1                             1
P               26     100      120       130
T                                                           -1     -87
W2             .14
X                        1
K        1      LL                                           1
K1           ROUTE HYDROGRAPH THROUGH LUCAS LAKE DAM
Y                                 1         1
Y1                                                       -719.5      -1
Y4   719.5     720   720.3    720.7       721    721.7   722.4   723.2   723.9   724.6
Y4   725.3
Y5       0      56     122      247       347      643     999    1521    2957    4997
Y5    7495
$A       0     1.5     6.5        8        10     13.5      17
$E     700     710   719.5      720       723      730     740
$$719.5
$D     723
$B      10      .5     710        1     719.5      723
```

73	K		L					1			
74	K1		RUNOFF CALCULATION FOR LUCAS DAM								
75	M	1	2	.13		.13	1			1	
76	P		26	100	120	130					
77	T							-1	-87		
78	W2		.1								
79	X			1							
80	K	1	L					1			
81	K1		ROUTE HYDROGRAPH THROUGH LUCAS DAM								
82	Y				1	1					
83	Y1	1						-729	-1		
84	Y4	729	729.7	730.4	731.1	731.8	732.5	733.1	733.8	734.4	735.1
85	Y4	735.7	736.4	737	737.8						
86	Y5	0	17	50	95	153	224	306	403	527	729
87	Y5	1520	2921	4456	6813						
88	$A	0	2	3	4.5	5.5	6.5				
89	$E	715	725	729	733.5	735	740				
90	$$	729									
91	$D	735									
92	$B	10	.5	720	1	729	735				
93	K		VL					1			
94	K1		RUNOFF CALCULATION FOR VALLE LAKE DAM								
95	M	1	2	2.16		2.16	1			1	
96	P		26	100	120	130					
97	T							-1	-87		
98	W2		.42								
99	X			1							
100	K	4	VL					1			
101	K1		COMBINE RUNOFF HYDROGRAPH AND THREE ROUTED HYDROGRAPHS								
102	K	1	VL					1			
103	K1		ROUTE COMBINED HYDROGRAPH THROUGH VALLE LAKE DAM								
104	Y				1	1					
105	Y1	1						-684	-1		
106	Y4	684	684.5	684.8	684.9	685.3	685.7	686.3	687.2	687.9	688.5
107	Y4	689	690	690.4	691.2	691.9	692.5				
108	Y5	0	14	79	129	280	530	1030	2030	3030	4030
109	Y5	5030	7996	9928	15407	21020	26765				
110	$A	0	2	12.5	32	34	42.5	44	57.5	94	131.5
111	$A	171									
112	$E	655	660	670	680	680.9	684	684.5	689	700	710
113	$E	720									
114	$$	684									
115	$D	689									

SUMMARY OF DAM SAFETY ANALYSIS

1		INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
	ELEVATION	827.50	827.50	827.70
	STORAGE	98.	98.	101.
	OUTFLOW	0.	0.	30.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	827.77	.07	102.	1565.	.48	15.92	6.58
.50	827.72	.02	102.	1411.	.35	8.25	7.25

SUMMARY OF DAM SAFETY ANALYSIS

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	782.50	782.50	785.20
STORAGE	45.	45.	64.
OUTFLOW	0.	0.	675.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	785.85	.65	70.	2942.	.60	15.83	7.42
.50	785.78	.58	69.	1620.	.58	9.08	8.08

PLAN 1 STATION LVM

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	2257.	690.8	16.17
.50	1250.	689.0	9.17

SUMMARY OF DAM SAFETY ANALYSIS

....	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	719.50	719.50	723.00
STORAGE	40.	40.	71.
OUTFLOW	0.	0.	1390.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	723.89	.89	80.	2928.	.42	15.75	0.00
.50	722.61	0.00	67.	1138.	0.00	15.83	0.00

SUMMARY OF DAM SAFETY ANALYSIS

....	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	729.00	729.00	735.00
STORAGE	17.	17.	41.
OUTFLOW	0.	0.	700.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	735.77	.77	45.	1667.	.35	15.69	15.67
.50	734.34	0.00	37.	515.	0.00	15.83	0.00

SUMMARY OF DAM SAFETY ANALYSIS

1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	684.00	684.00	689.00
STORAGE	431.	431.	681.
OUTFLOW	0.	0.	5030.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	691.92	2.92	861.	21173.	4.67	16.08	0.00
.50	690.37	1.37	762.	5798.	1.25	16.17	0.00

PERCENT OF PMF ROUTING
EQUAL TO SPILLWAY CAPACITY

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**********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION       JULY 1978
  LAST MODIFICATION  01 APR 80
**********************************
A1       MISSOURI DAM SAFETY
A2       VALLE LAKE DAM (MO.30438)
A3       PERCENT PMF
B      300       0       5       0       0       0       0       0      -4       0
B1       5
J        1       4       1
J1     .20     .25     .28     .30
K              UVM                                       1
K1       RUNOFF CALCULATION FOR UPPER VALLE MINES DAM
M        1       2     .25             .25       1                       1
P               26     100     120     130
T                                                       -1     -87
W2             .12
X                        1
K        1     UVM                                       1
K1       ROUTE HYDROGRAPH THROUGH UPPER VALLE MINES DAM
Y                                1       1
Y1       1                                          -827.5      -1
Y4   827.5   827.8   828.1   828.5   828.8   829.4
Y5       0      45     371    1118    1881    3924
$A       0     6.5      11      16      16    20.5    39.6
$E     813     820   823.7   827.5   827.7     830     840
$$   827.5
$D   827.7
$B      10      .5     815       1   827.5   827.7
K              LVM                                       1
K1       RUNOFF CALCULATION FOR LOWER VALLE MINES DAM
M        1       2     .19             .19       1                       1
P               26     100     120     130
T                                                       -1     -87
W2             .11
X                        1
K        2     LVM                                       1
K1       COMBINE ROUTED HYDROGRAPH AND RUNOFF HYDROGRAPH
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35   K       1     LVM                                                  1
36   K1            ROUTE COMBINED HYDROGRAPH THROUGH LOWER VALLE LAKE DAM
37   Y                                      1       1
38   Y1      1                                                     -785.2      -1
39   Y4  782.5   783.4     784   784.5     785   785.5     786
40   Y5      0      30     120     295     520     907    1573
41   $A      0       3     6.5       7       8      12      20
42   $E    765     775   782.5   783.4   785.2     790     800
43   $$  782.5
44   $D  785.2
45   $B     10      .5     770       1   782.5   785.2
46   K       1     LVM                                                  1
47   K1            STREAM ROUTING OF HYDROGRAPH TO VALLE LAKE DAM
48   Y                                      1       1
49   Y1      1
50   Y6    .08    .045     .08     684     726    6000   .0013
51   Y7      0     726     200     706     320     686     340     684     380     684
52   Y7    400     686     530     706     620     726
53   K             LL                                                   1
54   K1            RUNOFF CALCULATION FOR LUCAS LAKE DAM
55   M       1       2     .25             .25       1                       1
56   P              26     100     120     130
57   T                                                              -1     -87
58   W2            .14
59   X                       1
60   K       1     LL                                                   1
61   K1            ROUTE HYDROGRAPH THROUGH LUCAS LAKE DAM
62   Y                                      1       1
63   Y1                                                            -719.5      -1
64   Y4  719.5     720   720.3   720.7     721   721.7   722.4   723.2   723.9   724.6
65   Y4  725.3
66   Y5      0      56     122     247     347     643     999    1521    2957    4957
67   Y5   7495
68   $A      0     1.5     6.5       8      10    13.5      17
69   $E    700     710   719.5     720     723     730     740
70   $$719.5
71   $D    723
72   $B     10      .5     710       1   719.5     723
```

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 73   K              L                                                1
 74   K1          RUNOFF CALCULATION FOR LUCAS DAM
 75   M       1       2     .13             .13       1                             1
 76   P              26     100     120     130
 77   T                                                              -1     -87
 78   W2             .1
 79   X                       1
 80   K       1       L                                               1
 81   K1          ROUTE HYDROGRAPH THROUGH LUCAS DAM
 82   Y                               1       1
 83   Y1      1                                                    -729      -1
 84   Y4    729   729.7   730.4   731.1   731.8   732.5   733.1   733.8   734.4   735.1
 85   Y4  735.7   736.4     737   737.8
 86   Y5      0      17      50      95     153     224     306     403     527     729
 87   Y5   1520    2921    4456    6813
 88   $A      0       2       3     4.5     5.5     8.5
 89   $E    715     725     729   733.5     735     740
 90   $$    729
 91   $D    735
 92   $B     10      .5     720       1     729     735
 93   K              VL                                               1
 94   K1          RUNOFF CALCULATION FOR VALLE LAKE DAM
 95   M       1       2    2.16            2.16       1                             1
 96   P              26     100     120     130
 97   T                                                              -1     -87
 98   W2            .42
 99   X                       1
100   K       4      VL                                               1

101   K1          COMBINE RUNOFF HYDROGRAPH AND THREE ROUTED HYDROGRAPHS
102   K       1      VL                                               1
103   K1          ROUTE COMBINED HYDROGRAPH THROUGH VALLE LAKE DAM
104   Y                               1       1
105   Y1      1                                                    -684      -1
106   Y4    684   684.5   684.8   684.9   685.3   685.7   686.3   687.2   687.9   688.5
107   Y4    689     690   690.4   691.2   691.9   692.5
108   Y5      0      14      79     129     280     530    1030    2030    3030    4030
109   Y5   5030    7996    9928   15407   21020   26765
110   $A      0       2    12.5      32      34    42.5      44    57.5      94   131.5
111   $A    171
112   $E    655     660     670     680   680.9     684   684.5     689     700     710
113   $E    720
114   $$    684
115   $D    689
```

SUMMARY OF DAM SAFETY ANALYSIS

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	827.50	827.50	827.70
STORAGE	98.	98.	101.
OUTFLOW	0.	0.	30.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.20	827.75	.05	102.	1439.	.46	13.33	12.33
.25	827.70	.00	101.	1387.	.21	11.42	10.42
.28	327.70	.00	101.	1390.	.25	10.25	9.25
.30	627.70	.00	101.	1391.	.25	9.75	8.75

SUMMARY OF DAM SAFETY ANALYSIS

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	782.50	782.50	785.20
STORAGE	45.	45.	64.
OUTFLOW	0.	0.	675.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.20	785.82	.62	69.	1677.	.60	14.17	13.17
.25	785.75	.55	69.	1596.	.56	12.25	11.25
.28	785.75	.55	69.	1582.	.56	11.08	10.08
.30	785.75	.55	69.	1586.	.56	10.58	9.58

PLAN 1 STATION LVM

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.20	1311.	689.1	14.25
.25	1223.	688.9	12.33
.28	1219.	688.9	11.17
.30	1221.	688.9	10.67

SUMMARY OF DAM SAFETY ANALYSIS

1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	719.50	719.50	723.00
STORAGE	40.	40.	71.
OUTFLOW	0.	0.	1390.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.20	721.15	0.00	53.	412.	0.00	15.83	0.00
.25	721.43	0.00	56.	528.	0.00	15.83	0.00
.28	721.59	0.00	57.	595.	0.00	15.83	0.00
.30	721.69	0.00	58.	639.	0.00	15.83	0.00

AD-A104 962 PRC CONSOER TOWNSEND INC ST LOUIS MO F/G 13/13
NATIONAL DAM SAFETY PROGRAM. VALLE LAKE DAM (MO 30438), MISSISS--ETC(U)
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SUMMARY OF DAM SAFETY ANALYSIS

....	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	729.00	729.00	735.00
STORAGE	17.	17.	41.
OUTFLOW	0.	0.	700.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.20	731.97	0.00	27.	170.	0.00	15.83	0.00
.25	732.46	0.00	29.	220.	0.00	15.83	0.00
.28	732.73	0.00	30.	255.	0.00	15.83	0.00
.30	732.90	0.00	31.	279.	0.00	15.83	0.00

SUMMARY OF DAM SAFETY ANALYSIS

1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	684.00	684.00	689.00
STORAGE	431.	431.	681.
OUTFLOW	0.	0.	5030.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.20	688.20	0.00	636.	3525.	0.00	16.25	0.00
.25	688.60	0.00	658.	4230.	0.00	16.25	0.00
.28	688.89	0.00	674.	4604.	0.00	16.25	0.00
.30	689.07	.07	685.	5228.	.25	16.25	0.00